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The influence of uncertainty on a hydrological model and a sensitivity analysis to account for imperviousness in drainage analysis.

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Professional Practice Research Report

The influence of uncertainty on a hydrological model and a sensitivity analysis to account for imperviousness in drainage analysis.

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Abstract

Model outputs are always burdened with uncertainties which are unavoidable, when using inputs and parameters which are encumbered with their own uncertainties. Ideally, the exact uncertainty values about the model output are derived from sensitivity analysis. However, due to the complex nature of functions involved in models, uncertainty analysis can rarely be performed analytically. Hence, alternative methods are often applied, such as the first-order second-moment method (FOSM), the Monte Carlo approach, point estimate (PE) or parameter evaluation methods. The latter method is more practical, as they require a smaller number of inputs and computations (Chang, et al., 1995).

The following paper is a sensitivity analysis of the influence of uncertainty burdened parameters and their influence on the model outputs, as well as the sensitivity of the model to changes in one parameter – imperviousness – which may be associated with increasing urbanization. The model used for the study is the Watershed Bounded Network Model, and the catchment evaluated is located in south-east Australia. The initial model is very conservative, assuming no impervious cover over the surface.

The study consists of two separate parts: the performance parameter evaluation, which gives insight into the model's behaviour when changing the impervious cover extent, and the Two-Point Technique evaluation which gives a general overview of how the model reacts to parameters burdened with uncertainty. The analysis will be conducted for different intensity storms according to the Australian Rainfall and Runoff.

The results of both analyses showed, that increasing storm intensity leads to a decrease in the sensitivity of the model to input parameter uncertainties, as well as the model is less sensitive to changes in impervious cover. However, this may be also observed due to the rapidly increasing output value. Overall, based on this study it is possible to say that the initially provided conservative model is suitable to generate results indifferently of the assigned impervious cover extent in the investigated catchment. Combining the analysis conducted in this paper with a hydraulic model would provide the information on how high the water level would raise with an increase in peak discharge caused by increased imperviousness would. Finally, based on the Two-Point Technique, it is visible that the model is prone to uncertainties related to its input parameters however small they may be.

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1. Introduction

Hydrologic modelling plays an increasing role in the sustainability of human societies. The rapidly growing population leads to a swift expansion of urban areas. Urbanization brings change to the physical properties of the land, which in turn have an irredeemable effect on water storage, routing and overall water behaviour. Hydrologic modelling is a crucial step in sustainable water management, concerning not only urban areas but entire river catchments. Therefore, the aspect of urban cover, or more precisely, imperviousness is considered a significant parameter in any hydrological study. As any parameter, impervious cover extent is encumbered with uncertainty, which in turn, will have effects on the model's outputs and their uncertainty.

Increasing complexity of hydrologic models leads to a higher number of parameters needed to be estimated and longer computation time. In order to get a better understanding of how models work, sensitivity analysis methods should be used prior to applying the hydrologic models in practice. Since there is a large amount of sensitivity analysis methods available, it is crucial to understand and apply an appropriate method for the given model and data available. According to Loosvelt et al. (2013) a good modelling practice comprises both of evaluating the uncertainty in the model results as well as evaluating how much does each parameter contribute to the output uncertainty.

Model outputs are always encumbered with uncertainties which are unavoidable, when using input parameters that are encumbered with uncertainties of their own. Ideally, the exact uncertainty values about the model output are derived from uncertainty analysis. However, due to the complex nature of functions and their relations involved in models, uncertainty analysis can rarely be performed analytically. Hence, alternative methods are often applied, such as the first-order second-moment method (FOSM), the Monte Carlo approach, point estimate (PE) or parameter evaluation methods. The latter method is more practical, as it requires a smaller number of inputs and computations (Chang, et al., 1995).

1.1. Aim and scope of work

The aim of the following thesis is to examine the influence data uncertainty has on the precision in determining the overland flow throughout a particular catchment with a provided model. One of the main factors affecting overland flow is urbanization and the increased coverage of pervious surfaces with concrete. Hence, it will be the parameter the following study will focus on.

Objectives:

- Study the parameters used in the model, along with their uncertainties.
- Analyse peak flow values at a chosen point in the catchment given for different annual recurrence intervals (ARI).
- Evaluate how sensitive the provided model is to the parameter of imperviousness with the use of performance parameter evaluation.

- Evaluate how sensitive the model is to parameters burdened with uncertainties, especially the parameter of impervious surface based on a point estimate method – the Two-Point Technique.
- Establish if future urbanization may have a significant effect on the drainage analysis based on the provided model.

1.2. Motivation

The hosting company has been assigned to prepare a Condition Assessment, Hydrology and Concept Design report for an existing underline crossing culvert located in New South West, Australia. As the company wanted to keep the details of the project confidential, the watershed used in this study will be referred to as the 'catchment' further on. The scope of the project included a flood study and an estimation of design peak flows which in turn were calculated using a very rigorous hydrological model. The provided Watershed Bounded Network Model (WBNM) was very conservative, assuming zero impervious cover in the catchment and minimal losses. The study rose questions concerning how the impervious cover extent over the catchment may influence the runoff and flow in the drainage analysis. After some consideration, it has been noted that the further development of the catchment may lead to changes in some parameters, imperviousness in particular. The hydrological model generated was intended to be used further in hydraulic analysis to establish the potential impact, intense precipitation may have on a culvert structure located at the downstream part of the stream channel. The study aimed to assess whether the existing structure can withstand the potential flooding and not be overtopped. Since the provided model was very conservative, the impervious cover and losses became parameters that may have been encumbered with great uncertainty.

This created doubts concerning the reliability of the model and the need to conduct an analysis on what influence do different parameters, particularly imperviousness, have on the model's outputs. This further expanded the scope of the study to evaluate the effect several parameters have on the model; the case of losses calculation became significant to the analysis as well. The need for this analysis created the aim of the following paper. The objectives include observing adjustments that may be made to the provided hydrological model, conducting a sensitivity analysis on its parameters and account for urbanization in the drainage analysis of the particular catchment.

2. Hydrological Modelling in Australia

Australian urban catchments have separate sewage and stormwater collection systems and their soil types underlying the urban areas have large variations in infiltration characteristics. These two features of the Australian catchments were the main reason why in the past, catchments in Australia were not modelled with overseas computer models, which were set up for single collection systems (Dayaratne, 2000). Currently, this is not an issue anymore, as the overseas models may be adapted according to the required characteristics of the catchment. Despite that, many of the hydrological models existing today, are based on local, Australian models, despite some oversea models being successfully modified to be applied to the Australian conditions. Amongst popular urban drainage computer models used for design and analysis in Australia are ILSAX (ILLUDAS-SA with something extra (ILLUDAS-SA-eXtra)), RAFTS (Runoff Analysis and Flow Training Simulation), SWMM (Storm Water Management Model) and WBNM (Watershed Bounded Network Model).

2.1. Overland Flow models

2.1.1. ILSAX

The ILSAX model name comes from "ILLUDAS-SA with something extra (ILLUDAS-SA-eXtra), the Illinois Urban Drainage Area Simulator (Dayaratne, 2000), and is an American model adjusted for the Australian catchment conditions. ILSAX models the behaviour of a catchment and/or pipe system for real storm events and statistically based design storms. As in many rainfall-runoff models, ILSAX divides the catchment into sub-catchments based on the areas land use and other physiographic conditions. The sub-catchments are divided into three areas, i.e. the impervious, pervious and supplementary areas with an additional area that is not added to the runoff, like a swimming pool. ILSAX requires information on moisture content since it's an event-based model. Losses are calculated with two methods, i.e. losses subtracted from rainfall and losses subtracted from supply rate.

2.1.2. SWMM

The Storm Water Management Model (SWMM), a pipe drainage model, simulates urban runoff both in quality and quantity in separate and combined sewer networks. SWMM models all aspects of the hydrologic and quality cycles i.e. surface runoff, storage and transport through the drainage network. In the model, the catchment is divided into sub-catchments from which based on their characteristics, runoff is calculated. The sub-catchments are represented as spatially lumped, nonlinear reservoirs with a pipe routing their outflow. Each sub-catchment is divided into three sub-areas, the impervious area with depression storage, impervious area without depression storage and pervious areas with depression storage. The model uses nonlinear reservoirs as approximations of overland flow through each sub-area of a sub-catchment (EPA, 2016). Horton or Green-Ampt equations are used within the model to calculate infiltration through pervious areas.

2.1.3. RAFTS

The Runoff Analysis and Flow Training Simulation (RAFTS) model simulates runoff from natural channels, modified channels, pipes, retention and retarding basins or any combination of them. RAFTS model's runoff hydrographs at points defined throughout the catchment. Like in most rainfall-runoff models, RAFTS divides the catchment into sub-catchments which are in turn then divided into ten sub-areas. The division is done based on isochrones or the lines of equal travel time. Laurenson's runoff routing procedure is used to generate outflow hydrographs for each sub-catchment. Pervious and impervious areas are modelled separately, however RAFTS does not consider connected impervious areas and supplementary areas separately as do ILSAX and SWMM. RAFTS uses Manning's equation to determine pipe flow, for flood routing through pipes – the Muskingum method and for retarding and retention basins – Puls' level pool routing procedure. In places where data is lacking, the program offers a channel lagging procedure, where the hydrograph is lagged for a determined amount of time, with no attenuation. Lag time is calculated based on the velocity computed from the Manning equation (Dayaratne, 2000).

2.1.4. WBNM

The Watershed Bounded Network Model (WBNM) is a lumped, event based, nonlinear runoff routing model. It allows modelling of both small and large catchments, which are divided into several sub-catchments, treated as separate storage elements within the model. Overland flow computed for each sub-catchment is modelled by a nonlinear reservoir with lag time. Overall, WBNM offers three different methods for channel routing calculation: nonlinear routing, Muskingum routing and time-lag method. Each sub-catchment is divided into impervious and pervious areas, each with a separate rainfall loss used to compute excess rainfall. Several alternative loss models are available within the model (initial loss-constant loss rate, initial loss-loss rates varying in steps, initial loss-runoff proportion, Horton continually varying loss rate and Green-Ampt varying loss) (Boyd, et al., 2012).

Table 1 compares the main features of the listed models. With a wide range of hydrologic and hydraulic models available, one must be chosen to fit the catchment of interest, based on the available data, computation time and model complexity required. For this particular study, a WBNM has been chosen as the suitable model.

Table 1: Comparison of the main features of the four models: ILSAX, SWMM, RAFTS and WBNM.

Model Name	Model Type	Impervious and Pervious Area Calculation	Loss Model Type	Overland Flow Routing Method	Pipe Routing Method	Water Quality Parameter Simulation	Output Data
ILSAX	Event	Separate	Horton	Time-area	Manning's equation	Not available	Hydrographs at each pit
SWMM	Event or continuous	Separate	Horton / Green-Ampt	Nonlinear reservoir	Kinematic wave	Available	Hydrographs at each pit
RAFTS	Event / continuous	Separate	Philip's equation / ARBM model	Laurenson's runoff routing procedure	Muskingum method / Time-lag	Not available	Runoff hydrographs generated at defined points
WBNM	Event	Separate	initial loss-constant loss rate / initial loss-loss rates varying in steps / initial loss-runoff proportion / Horton continually varying loss rate / Green-Ampt varying loss	Non-linear reservoir with lag time	Nonlinear / Muskingum / Time-lag	Not available	Runoff hydrographs generated at defined points

2.2. Losses in drainage analysis

Increased streamflow in Australia is mainly caused by intensive rainfall conditions. Due to the temperate climate, snow and ice melts have a much smaller impact on runoff. Rainfall that falls onto the ground may be converted into runoff based on several factors. Those include the infiltration capacity, land cover, type and saturation of the soil. Typically, the majority of rainfall is intercepted by regionally rich vegetation, stored in surface depressions, evaporated or infiltrated to become groundwater or soil moisture. The remaining rain water is converted into stream flow. However, in conditions where the catchment is saturated with water, the losses are greatly reduced, and the majority of rainfall is converted into streamflow. Runoff process then consists of infiltration excess runoff, saturation excess runoff, sub-surface stormflow and impervious runoff.

Natural catchments may be covered with impervious areas such as rocky outcrops and slopes. Urbanized catchments are covered with roofs, car parks, roads, and other paved surfaces. The majority of rain that falls on impervious surfaces is converted into runoff, as there is little place for the water to be intercepted, stored and lost. Hence, urbanization has a major contribution in increasing runoff volume, flood frequency and magnitude (Ladson, et al., 2019). Studies have shown, that urbanization leads to a ten-fold increase in peak flows during floods ranging from one to four exceedances per year (EY) with impacts diminishing in terms of large floods (Tholin & Keifer, 1959), (ASCE, 1975), (Espey & Winslow, 1974), (Hollis, 1975), (Cordery, 1976), (Ferguson & Suckling, 1990).

Heavily forested catchments rarely have surface runoff since soil infiltration rates are rarely exceeded by rainfall. In turn, water in these catchments may be routed through these areas by subsurface flows. According to the Australian Rainfall and Runoff (ARR), losses in flood hydrology refer to any rainfall that is not converted into quick-flow. After subtracting the losses, the remaining rainfall is called as the rainfall excess which is the quick-flow produced on the catchment. Losses values may be estimated based on historic events, where the volume of runoff, catchment area and rainfall depth were all measured. Based on this, losses may be estimated for a range of catchments (Hill & Thomson, 2019).

The model chosen for modelling a catchment should be greatly based on the data that is available. Complex models may require too much approximation when data is lacking, presenting poor predictive ability, while simple models may not use the available data to the fullest. The popularity of simple hydrologic models has grown significantly as their complexity matches reasonably with the limited data which is easily available for most catchments these days. Hydrologic models usually incorporate simple loss models, based on two parameters, the Initial Loss (IL) and the Continuous Loss (CL). According to the Australian Rainfall and Runoff, the initial loss – continuous losses model is the most effective when it comes to modelling both urban and rural catchments in Australia. The model takes into account a constant value for initial losses for each sub-catchment along with a constant value of continuous losses for the given flood event (Green, et al., 2019). For urban catchments, the IL and CL are then divided into pervious (IL_{perv}), impervious (IL_{imp}) and indirectly connected areas. However, in WBNM, CL remain the same for both pervious and impervious areas and indirectly connected areas are not considered separately.

Due to catchment process and efficient drainage, flood runoff from urban areas is larger than from rural catchments. Urban area runoff is generated from impervious surfaces, which are characterized by low interception losses due to few green areas and small depression storage areas caused due to smooth surfaces with low infiltration. According to Boyd et al. (2012) who studied 763 rain events in urban areas, initial losses for impervious surfaces were lower than 1 mm. The average IL weighted over the number of events was 0.62 mm. 70 % of the urban catchments studied had IL equal to or less than 1 mm.

The catchment used for this study is of relatively small size and the divided sub-catchments have limited spatial variation in rainfall and loss characteristics. Hence, a model such as WBNM is suitable to use to treat each sub-catchment as a homogenous unit (lumped). Linking the outputs of each sub-catchment allows to create a semi-distributed catchment model. The lumped flood hydrograph estimation model is limited in its application, as it may be applied only to catchments with uniform rainfall spatial distribution, loss and baseflow characteristics and to catchments without significant artificial storages. Considering the black box approach of a lumped model, observed flood hydrographs are required for calibrating the model.

Semi distributed models, also known as Node-Link Type models are one of the most often used methods for flood hydrograph estimation techniques in Australia. The Node-Link Type models allow to represent the catchments features in a conceptualized form. The conceptualizations require certain lumping in terms of the processes modelled and spatial averaging of inputs. The runoff routing process in a semi distributed model, each sub-area receives excess input that is converted into a runoff hydrograph at the node that represents the area. Then, the hydrograph is routed through each area towards the catchment outlet. Dividing the catchment into sub-catchments (sub-areas), provides a simplified but physically based representation of the spatial features of the catchment. A relatively small number of areas is used for this, from ten to a hundred sub-catchments. These are usually generated based on the topographic features which control the movement and storage of flood waters. Land features and structures must be specifically delineated in the model (Hill & Thomson, 2019).

Based on observations of natural catchments during storm periods, two types of storm runoff mechanisms have been proposed by the ARR: saturated overland flow and throughflow. Saturated overland flow occurs when a part of the surface horizon of the soil is saturated and the saturated zone level is built above the soil horizon. Throughflow describes the flow of water that has infiltrated the soil and percolates quickly through macropores, cracks, root holes and others to reach a stream channel. However, no practical methods have been developed to model both different runoff processes. Existing models usually assume uniform or average conditions to simplify the complexity of these physical processes (ASCE, 1975).

2.2.1. Loss models applied

Loss models may be broken down into three types, the empirical models, simple models and process models. Empirical models are designed to ensure direct runoff and rainfall are in equilibrium, maintaining at the same time factors that influence values characteristic to an individual catchment. Simple loss models aim to simplify and quantify a portion of the processes, where for instance all losses may be assumed to relate to infiltration like Hortonian Infiltration Models. Finally, Process models attempt to represent the complex behaviour of losses such as flow through soil layers over the entire catchment surface, through continuous simulation, but since these are not extensively used in Australia, they will not be evaluated further on.

Empirical Models aim to represent flow, giving less focus to the loss processes themselves. Majority of rainfall excess models are empirical models, where the initial losses occur at the beginning of the storm. The initial losses collected at the beginning include interception losses, depression storage and infiltration. This is followed by continuing loss rates, which are applied throughout the remaining part of the storm. Such models are consistent with the idea of runoff produced by the excess rainfall that has not been intercepted. Initial Loss – Continuing Loss models or the Initial Loss – Proportional Loss are some of the examples of empirical loss models.

Initial Loss – Continuing Loss models are characterized with a simple conceptual nature which often leads to challenges involving estimating continuing loss directly from rainfall and streamflow recorded. If the CL are calculated from the water balance of runoff volume minus the initial loss and divided by the event duration, the loss rate may be underestimated. Sometimes the rainfall will be smaller than the continuous loss rate and therefore the total value will not be used up. This loss model is the one applied as default in the provided WBNM.

The Initial Loss – Proportional Loss model considers a set value or percentage of the rainfall is lost at each timestep after the initial losses are subtracted from the rainfall. Losses during an event may differ based on temporal patterns of rainfall. Amongst other Simple Models, there is the Variable Continuing Losses method, SCS Curve Number method or the Probability Distributed Storage Capacity Models.

Simple Models attempt to incorporate infiltration of the rainfall to the soil taking into account the soils properties, moisture conditions, layers of soil, rainfall intensity, vegetation cover, soil slope and land use (Siriwardene, et al., 2003). The water movement through the soil may be described with more and less complex equations. The Horton Model provides an estimate of losses due to rainfall infiltration into pervious surfaces based on a decreasing continual loss. It is described by equation 1 below.

$$f_{t=}f_c + (f_0 - f_c)e^{-kt} \tag{1}$$

Where:

f_t – infiltration capacity [mm/h]

f_c – minimum or ultimate value of infiltration capacity [mm/h]

f_0 – maximum or initial value of infiltration capacity [mm/h]

k – decay coefficient [per hour]

t – time from the beginning of the storm [h]

The Green-Ampt model is an infiltration model based on approximate theory developed by Green and Ampt (1911). The model utilizes Darcy’s law. Studies have shown that although the Green-Ampt model provides superior results compared to other Simple Models when applied at a catchment scale. However, compared to empirical models, the results were not on average superior.

2.2.2. Estimating loss values

There may be several approaches to estimating the appropriate loss values for a catchment and these include the empirical analysis of at-site rainfall and flow data; gathering data from regional analysis; or assimilating design values based on independent flood frequency research. Each of the different approaches for Estimating Loss Values has its advantages and disadvantages. Empirical analysis of at-site rainfall and flow data is greatly advantageous as it accounts for the catchment of interest characteristics and is directly relevant to the location of interest. However, this method is only applicable for catchments without any diversions or regulations. It is difficult to select an unbiased sample of events which is usually small and makes it difficult to explore the distribution of loss values. Regional analysis allows for a distribution of a larger sample of values and relationships with different characteristics to be explored, along with giving a more selective choice of data sets for analysis. The drawback of this method is that it is difficult to link loss values to rainfall and catchment characteristics and considerable effort is required in this method. Additionally, Regional Information does not guarantee that the loss values will give unbiased flood estimates. Finally, assimilation of design values with independent flood frequency estimates gives results which when combined with other design inputs, produces loss values which are unbiased estimates. The nature of design rainfall is essentially accounted for by the loss values (I.E., 2019). A great drawback to the method is that additional uncertainty is introduced when sufficient streamflow data is not available.

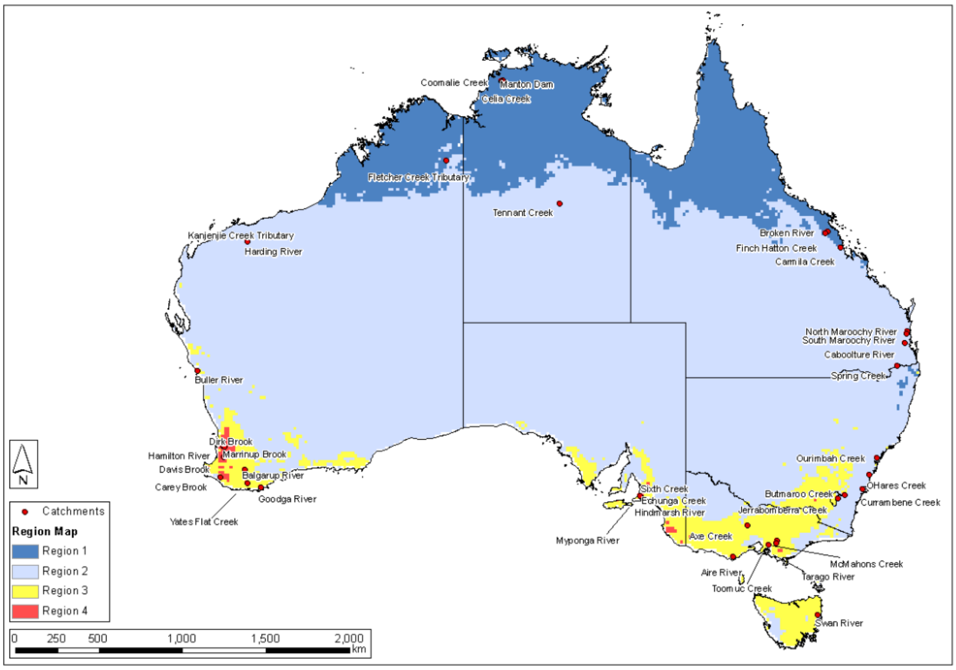


Figure 1: Regions Adopted for Loss Predication Equations (Hill & Thomson, 2019).

IL and CL values for rural catchments may be calculated with the use of Prediction Equations. The prediction equations were used to develop recommended loss values from the Australian Water Resource Assessment – Landscape (AWRA-L) developed by the Bureau of Meteorology (BoM) and Commonwealth Scientific and Industrial Research Organisation (CSIRO) (Frost, et al., 2015). The model AWRA-L simulates the water balance on a continental scale with spatial resolution of 5 x 5 km. The outputs produced by the model include soil moisture, runoff, actual and potential evapotranspiration, deep drainage and leaf area index (LAI) (Smith, et al., 2016). Different regions are split according to different soil characteristics for which in turn, the model produces loss results. Investigating soil characteristics and dividing regions based on soil moisture content is considered a better foundation for regionalization compared to basing it on rainfall alone. Soil moisture takes into account both the catchment storage and the climate conditions lasting in the area.

The four regions defined for the assessment are visible in figure 1. Region 1 and 3 are characterized by primary summer and winter dominant regions. Region 2 presents a uniform climate over a large area; however, the majority of data is obtained from the eastern portion of the region. Region 4 represents the catchments in the south-west of Western Australia. Figure 2 presents the seasonality of average gridded soil moisture for each of the regions. The investigated catchment is located in Region 2, with uniform, climate characteristics (I.E., 2019).

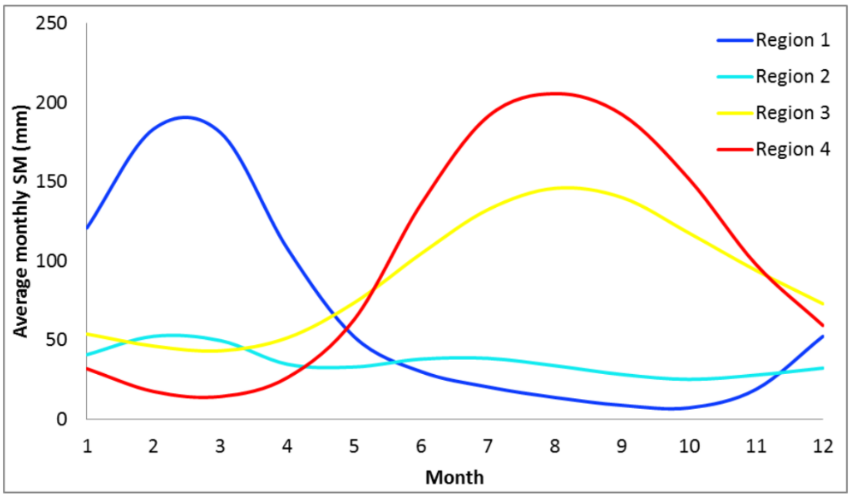


Figure 2: Seasonality of Average Gridded Soil Moisture in Each Defined Region (Hill & Thomson, 2019).

Prediction equations for initial losses and continuous losses were developed based on multilinear regression. Australian Rainfall and Runoff recommends loss values based on the prediction equations obtained. The losses distribution is visible in figures 3 and 4. The particular values for a catchment may be accessed via the ARR Data Hub online.

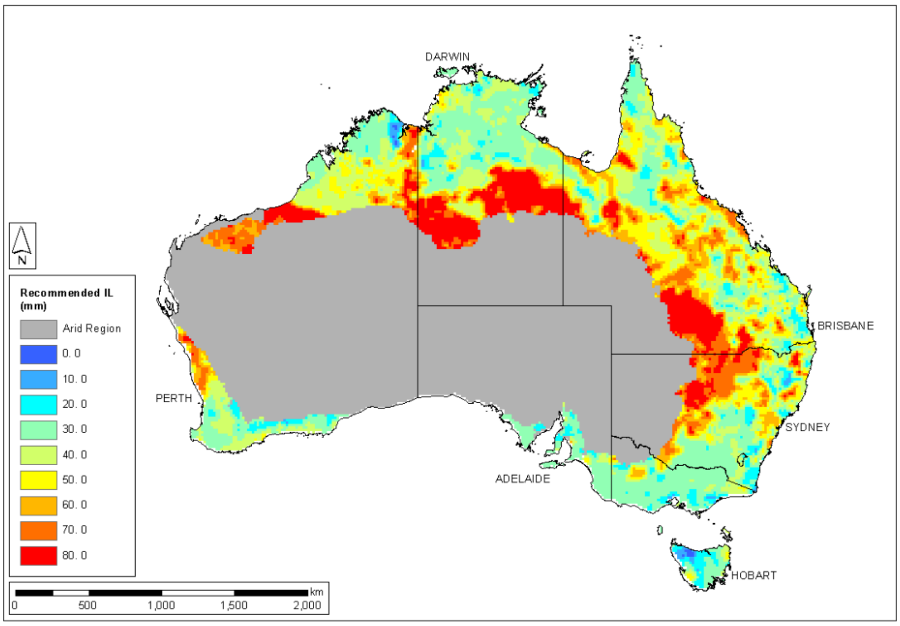


Figure 3: Recommended Median IL [mm] (Hill & Thomson, 2019).

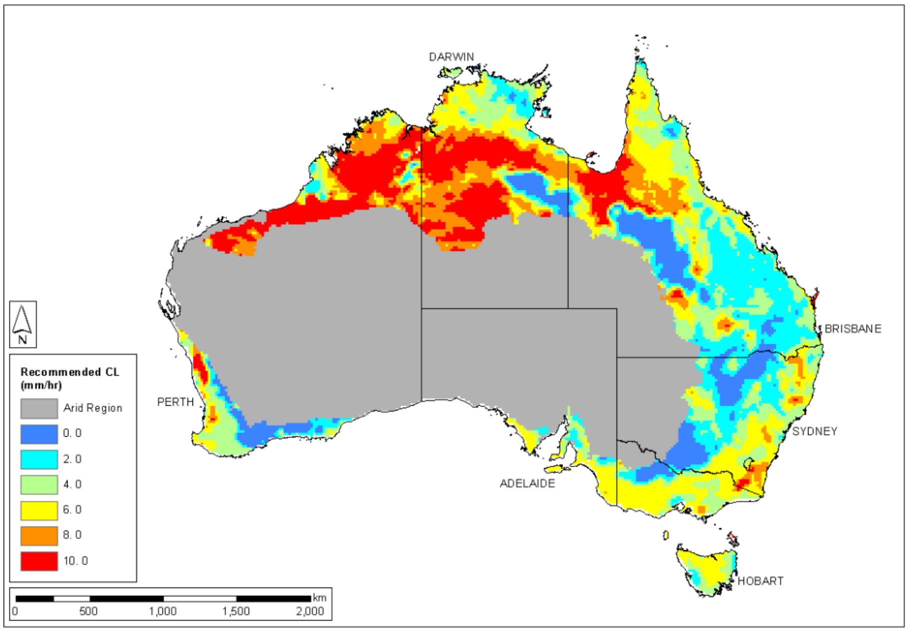


Figure 4: Recommended Median CL [mm/hr] (Hill & Thomson, 2019).

The recommended storm initial losses in the ARR are between 20 – 60 mm, while for continuous losses between 1.4 - 8.3 mm / hr and between 1-2mm for effective impervious area, where CL for effective impervious area (EIA) may be assumed as zero. The values given represent the annual median values, not taking seasonality into consideration. However, due to season variation throughout the year, rainfall intensity, evapotranspiration and vegetation cover varies depending on the seasonal hydrology. Research into seasonal variation should be done when there is a strong variation in the flood producing mechanisms due to seasonal changes or when there is a requirement to assess risk for a particular period within the year. Figure 5 presents the variation in seasonality depending on different parts of Australia. According to studies done by Phillips et al (2014), the Australian east coast initial losses are equal to 33 mm, but the values vary from study to study.

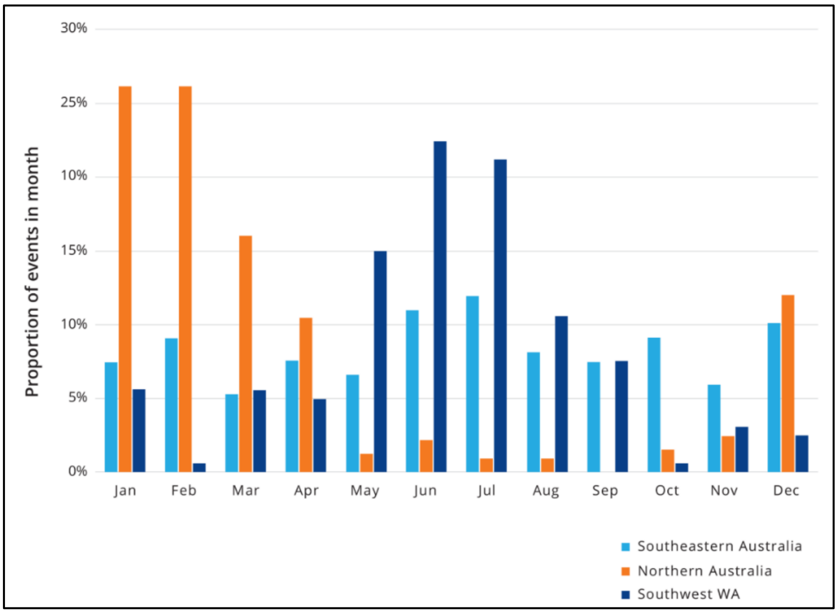


Figure 5: Seasonal Distribution of Events Analysed (Hill & Thomson, 2019).

Figure 6 presents the change in the initial loss values for the south-eastern Australia throughout the year. It is visible, that throughout the year, the values remain relatively constant.

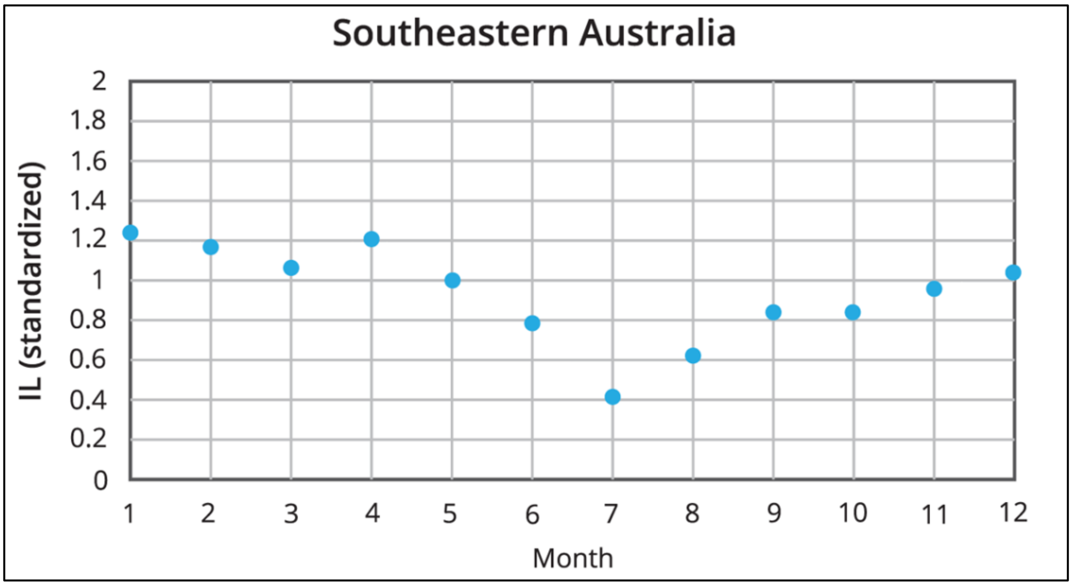


Figure 6: Seasonality of Standardized Storm Initial Loss Values for Different Regions in Australia (Hill & Thomson, 2019).

The two major categories of design flood estimation are streamflow-based methods and rainfall-based methods. Out of these two, the rainfall-based methods are used more often as rainfall data is usually more easily available compared to flood data. Physical catchment characteristics are easily consolidated into rainfall-based methods, which require several input parameters to obtain a design flood event from a design rainfall event. Amongst the required input parameters are the design losses, which define the amount of water that is trapped or absorbed and does not contribute to direct surface runoff (I.E., 2019).

The most commonly used loss model in Australia is the Initial Loss – Continuing Loss Model which is a lumped conceptual model and is used in design flood estimation as it is simple and capable to approximate catchment runoff behaviour in design flood estimation. Losses in this model are divided into Initial Loss, which are the amount of water that is the initially abstracted and Continuous Loss which are the average amount of rainfall abstracted throughput the event (Rahman, et al., 2016). Standard values for both Initial Loss and Continuous Loss have been recommended by the ARR (I.E., 2019).

Several studies done by Waugh (1991), Hill and Mein (1996), Rahman et al. (2002), Ilahee et al. (2001) have indicated that the recommended design values have limitations to them. The studies presented how regional flood methods utilise regional loss values to benchmark flood estimates with different methods. Rahman et al. (2016) took it upon themselves to derive the IL and CL values for selected catchments in New South Wales (NSW) in Australia that may be applied both in Design Event Approach and the Joint Probability Approach / Monte Carlo Simulation to design flood estimation. It has been suggested that the results obtained from the study may be applied to the overall region of NSW. Based on the studied events, the IL average value has been found at 21.84 mm with a standard deviation of 18.88mm and a coefficient of variation of 0.86, while CL average value was found as 1.20 mm/h with a standard deviation of 1.20 mm/h and a coefficient of variation of 0.87 (Rahman, et al., 2016).

3. Watershed Bounded Network Model

3.1. Theoretical Background

The Watershed Bounded Network Model has been developed originally to model natural catchments but has several other options which allow substantial flexibility in modelling complex flood cases. In order to create a realistic representation of the ongoing processes in the catchment, WBNM has built in lag relations based on the catchment's geomorphological features. The entire catchment is divided into sub-catchments or sub-areas by identifying the main stream and its major tributaries around which watershed lines (surface contours) are drawn based on where the water drains. Sub-catchments that contain the main stream and lead to the outlet route excess rainfall to produce a flood hydrograph and route runoff from the more upstream areas through the stream channel (Boyd, et al., 2012).

3.1.1. Flow routing within the model

Each sub-catchment may be composed of the following several elements: a stream channel, pervious surfaces, impervious surfaces, local structures (onsite detention storage collecting local runoff) and outlet structure (storage reservoir/flood detention basin on the main channel). Based on the first three elements listed, the flow is routed as follows.

Firstly, in any given sub-area, the top of the stream channel takes all the previously generated hydrographs from the previous sub-catchments. These hydrographs are then summed and routed through the stream channel of the given sub-area according to equations 2 and 3. A hydrograph is generated at the bottom of the channel. The pervious portion of each given sub-catchment takes the rainfall hyetograph, subtracts rainfall losses from it and routs the excess rainfall according to equation 2 to produce a hydrograph from pervious surfaces. The impervious portion of the given sub-catchment takes the rainfall hyetograph and subtracts the initial losses and routes the excess rainfall to produce a hydrograph from impervious surfaces. Additionally, a reduced lag parameter is automatically calculated for the runoff generated from the impervious surfaces.

Conservation of mass:

$$I - Q = dS/dt \tag{2}$$

Where:
I – inflow rate at time t [m^3/s]
Q – outflow rate at time t [m^3/s]
S – volume of water stored on catchment surface at time t [m^3]
t – time [s]

Stored volume related to the outflow discharge:

$$S = 60 \cdot K \cdot Q \tag{3}$$

Where:
K – lag time between centroids of inflow and outflow hydrographs [min]
Q – outflow rate at time t [m^3/s]
S – volume of water stored on catchment surface at time t [m^3]

All the generated hydrographs are added together at the end of the stream channel, along with any hydrographs directed to the bottom of the given sub-area from outlet structures of upstream sub-catchments and external hydrographs to produce a single hydrograph at the bottom of the sub-catchment. Based on hydraulic considerations and recorded rainfall and flood hydrograph data, as stated by Askew (1968, 1970) lag time K decreases as flood discharges increase, allowing for a nonlinear model. Lag time K depends on the size of the subarea. If the value remains constant for different magnitudes of floods, the model would be considered as linear (Askew, 1970) (Askew, 1968).

WBNM separately calculates hydrographs for flow through the main channel stream and overland flow. Flow velocities and lag times differ for overland and channel flow. Additionally, this allows to modify the sub-catchment properties such as imperviousness, separately from changing the characteristics of the stream channels. Despite the lag time for stream flow and overland flow is calculated separately, both equations contain the same parameter called lag parameter, hence only one vale is required for both (Boyd, et al., 2012).

3.1.2. WBNM Concepts

WBNM is a simple model, which aims to minimize the required parameters used for its set-up, so that it may be suitable for catchments with limited data available. Therefore, although it considers the same parameters as other models do, it has several simplifications introduced into it. These will be evaluated further on.

1. Runoff lag time is defined as the difference between the centroids of excess rainfall hyetograph and the surface runoff hydrograph. It depends on physical properties of the catchment such as area, shape, slope (for natural catchments), impervious surfaces and stream channel modifications. Lag relations developed for sub-catchments and catchments may be applied to both, as they are geomorphologically and hydrologically similar. The lag relations used in WBNM have been developed by Askew (1968, 1970). Equation 4 prescribes lag time for each sub-catchment, which applies to the transformation of excess rainfall into a surface runoff hydrograph. The equation presents a nonlinearity component, where when lag decreases, discharge increases, and that lag time is proportional to travel distance. With decreased lag times, the produced hydrographs occur in a shorter time base, the peak discharge is larger and the time to its rise and fall is smaller.

   Overland Flow Lag Time:

   $$Overland\ Flow\ Lag\ Time = Lag\ Parameter \cdot A^{0.57} \cdot Q^{-0.23} \tag{4}$$

   Where:
   A – area [km^2]
   Q – outflow rate at time t [m^3/s]
   Lag parameter – [h]

2. Stream channel flow lag relations are calculated separately in WBNM. Runoff from upstream sub-catchments flows through the main stream channels. Lag time for flow through a stream channel should be related to its length. Boyd et al. (1979,1987) studied the relations between the channel length and the chosen sub-catchment area and the channel lag time and sub-catchment area. These relations led to creating the function as presented in equation 5, where 0.6 has been decided to be an appropriate factor to hydrograph routing in the stream channels (Kemp & Daniell, 1995) (Jenkins, 1997).

   Stream Channel Lag time:

   $$Stream\ Channel\ Lag\ Time = 0.6 \cdot Lag\ Parameter \cdot A^{0.57} \cdot Q^{-0.23} \tag{5}$$

Where:
A – area [km^2]
Q – outflow rate at time t [m^3/s]
Lag parameter – [h]

The average Lag Parameter value is estimated around 1.60-1.80 for the catchments observed in the studies, hence the recommended value to use for the parameter is near 1.6. The results generated may preferably be compared to recorded hydrographs to confirm the validity of the lag parameter. “The same value of the Lag Parameter should be used for all subareas (global value), unless there is good evidence for varying it.” (Boyd, et al., 2012).

In terms of natural catchments, according to studies done by Boyd et al. (2012) and Askew (1968, 1970) the dominant factor for routing flow is the catchment area. “Catchment shape and slope are strongly correlated with catchment area, indicating that they need not to be considered in many cases.” (Boyd, et al., 2012). Based on the area of the catchment and the required modelling detail, the number of sub-catchments is established. The condensations introduced in WBNM (equations 4 and 5) decrease the requirement for data - stream lengths and slopes are not needed, only area of each sub-catchment. Additionally, having a fixed value of nonlinearity and a set factor of 0.6 in flow routing, only a single value of the lag parameter needs to be established for each sub-catchment. Studies have shown that a similar value of the Lag Parameter may be applied to a wide range of different catchments with different floods (Boyd & Cordery, 1989), (Sobinoff, et al., 1983), (Webb & O'Loughlin, 1981) . Askew (1968, 1970) studied the effect of nonlinearity on flood studies in natural catchments. The studies found that the nonlinearity value does not vary much in-between catchments and a value of -0.23 is adopted on average. The nonlinearity causes the lag time to decrease as the discharge and velocities increase.

3. Studies on slope conducted by Askew (1969, 1970) have shown that slope was not a significant factor when evaluating lag times for natural catchments. The catchments area is the main factor that influences lag time, while catchment shape and slope have a significantly smaller impact on this parameter. Slope has been considered as having a small impact on the catchments lag time for several reasons.

For catchments of different size, catchment area may differ over a magnitude of five orders, while the difference in slope for different catchments is in a much smaller range, usually up to one order difference. This indicates a much lower relation between slope and lag time equations when developed.

Next, slope cannot be measured as accurately as area and it is often dependent on the map scale used. This leads to greater uncertainty associated with the adopted slope value. Studies have shown that area and slope are correlated – steep slopes will be located in smaller subareas of the catchments. The correlation is strong especially when looking at large catchments, however the same is observed in terms of different sub-catchments.

Hence, the relation between lag time and catchment area may be assumed to take slope into consideration. Lag relations for stream channel flows depend on the velocities within the streams. The travel times and flow velocities have been shown to remain constant throughout the stream, both in steep and flat reaches (Leopold, et al., 1964) (Pilgrim, 1997) (Pilgrim, 1982). This is due to the compensation for a decrease in slope by an increase in depth and hence hydraulic radius. This indicates that once again, slope is not a dominant parameter used for determining lag time for stream channel flow.

Since lag relations used in WBNM require only area, the data requirements are greatly simplified. Practice has shown that the results generated by WBNM including only area, are good and accurate.

4. Routing hydrographs in stream channels occurs when the given sub-catchment is indicated as a flow path. The hydrograph placed at the top of a stream channel is routed to its bottom according to three available options: nonlinear, Muskingam or with routing with delay.

- Nonlinear routing is chosen as the default option in WBNM for natural catchments and applies a modified form of equation 6. In case of natural catchments, the Stream Lag Factor parameter is equal to one. If the channel is modified in a way that flow velocities and lag time change, the Stream Lag Factor may be modified to achieve the desired effect. Guidelines for the Stream Lag Factor are provided in the “Details of the Theory used in WBNM”.

  Modified stream channel lag time:

$$Stream\ Channel\ Lag = Stream\ Lag\ Factor \cdot 0.6 \cdot Lag\ Parameter \cdot A^{0.57} \cdot Q^{-0.23} \qquad (6)$$

Where:
A – area [km^2]
Q – outflow rate at time t [m^3/s]
Stream Lag Factor – [h]
Lag parameter – [h]

- In the time delay option, the routed hydrograph is delayed by the time it takes for it to pass through the stream channel. No other modifications are added, the time delay is provided in minutes.

- Applying Muskingum Routing to the hydrograph at the bottom of the stream channel gives it attenuation and translation. The Muskingum parameter K (minutes) and parameter X (range 0 to 0.5) must be specified. It is also possible to apply the Muskingum-Cunge routing, which includes the stream channel properties into the calculations.

When the Lag Parameter, the Stream Lag Factor or the sub-catchment area is set as zero, no hydrograph routing occurs, and the hydrograph generated at the bottom of the stream channel is the same as the one on top of the sub-catchment.

5. The rainfall hyetograph for each sub-catchment is calculated based on the rainfall hyetographs available at rain gauges specified by the user. The input rainfall in WBNM is converted to mm/hour which is further then used as the basic unit. The rain gauges may be specified in two ways.

- Firstly, the Thiessen weighting factors may be specified for each rain gauge which contributes to each sub-catchment. The hyetograph of each sub-catchment is then weighed against the sum of all rain gauge hyetographs. The sum of all the Thiessen weights is set to one as a default, however the user may choose to change this value.

- The second option of rain gauge weight calculation is to use the coordinates of rain gauges and the coordinates of the centres of each sub-catchment area to assign weight to each rain gauge. WBNM locates the nearest rain gauge to a sub-catchment and associates its temporal pattern. The total depth of the storm event for a chosen sub-catchment is the weighted depth of all the surrounding gauges. The weight is calculated based on the inverse square distance of each rain gauge in reference to the sub-catchments area centre. Rain gauges located far away will carry a small weight.

6. Calculating Hydrographs from Pervious Surfaces: every sub-catchment with area greater than zero has the process of calculating hydrographs from pervious surfaces invoked. The losses from pervious surfaces are firstly subtracted from the sub-catchment's rainfall hyetograph. Losses in WBNM may be calculated according to four methods.

- Initial loss-continuing loss rate method – a global value for initial losses and a continuous loss rate are specified for all the sub-catchments or individual values are provided for specific sub-catchments.

- Initial loss – runoff proportion method – global values or values associated with particular sub-catchments are provided for initial losses and a runoff proportion in the range or 0-1.0 is given.

- Horton infiltration – global values or values associated with particular sub-catchments are provided for maximum or initial value of infiltration capacity (F_0), minimum or ultimate value of infiltration capacity (F_c) and decay coefficient (k) parameters.

- Time varying rainfall losses – for each period in a rainfall hyetograph, a different continually time varying loss rate may be specified. The same time varying loss rates are applied to all sub-catchments, making them global values.

Hydrographs from pervious surfaces are calculated based on the values of excess rainfall hyetograph using equation 3 for the lag time. If the lag parameter value is equal to zero, the hydrograph is not routed, and the pervious hydrograph is equal to the excess rainfall hydrograph from the previous surface. If the area of the sub-catchment is equal to zero, the pervious hydrograph is composed of zero values as well.

7. Hydrographs from impervious surfaces are generated from the impervious portion of the sub-catchment if the sub-catchment area is greater than zero and the impervious extent of this area is also more than zero. Rainfall losses associated with impervious surfaces are firstly subtracted from the sub-catchment's rainfall hyetograph. These are taken as initial losses and continuous losses are assumed to be zero. A modified lag relation is used to route excess rainfall hyetographs to create impervious surface hydrographs. Since flow velocities are greater on impervious paved surfaces compared to pervious, grassed surfaces. Based on studies conducted by Espey et al. (1997), Rao et al. (1972), Aitken (1975), NERC (1975) and others, lag times for impervious surfaces are taken as one tenth of those for paved surfaces. Some studies indicate that linear routing is appropriate to apply on impervious surfaces. Based on the analysis of multiple urban catchments in Australia (Bufill & Boyd, 1992), (Boyd & Milevski, 1996), the Watershed Bounded Network Model uses a linear routing method for impervious surfaces along with a reduced lag time. This relation is presented in equation 7. If the impervious surfaces have a small area, they will have a small lag time and the hydrograph generated from this surface will be similar to the excess rainfall hyetograph. WBNM splits each sub-catchment into pervious and impervious areas. If one wants to divide the sub-catchment into impervious, supplementary impervious and pervious surfaces, the pervious and supplementary impervious surfaces should be considered together, and an average rainfall loss should be applied to them.

Impervious Surface Lag time

$$Imperv\ ious\ Surface\ Lag = Impervious\ Lag\ Factor \cdot Lag\ Parameter \cdot \left(\frac{A \cdot IMP}{100}\right)^{0.25} \quad (7)$$

Where:
A – area [km^2]
IMP – impervious percentage [%]
Lag parameter – [h] value between 1.30 - 1.80
Impervious Lag Factor – [h] recommended 0.10

In cases where the Lag Parameter or the Impervious Lag Factor are set to zero, no routing occurs, and the hydrograph generated from the impervious surface is equal to the impervious excess rainfall hyetograph. If the area of the sub-catchment is equal to zero, the pervious hydrograph is composed of zero values as well. Calculating the pervious and impervious hydrographs separately in WBNM allows for considerable flexibility in modelling catchments.

3.2. Case study

3.2.1. The catchment

The catchment studied is located in the north east part of New South West state, Australia. The state comprises of a multitude of geographical structures, having a 1460 km long coastal strip, from a subtropical Northern Rivers region to the chilly far south coast. The terrain elevation ranges from sea level to 1000 meters at the Great Dividing Range mountains. The remain part of the state is covered with plains, which cover over two thirds of the state. The majority of the population is distributed in the coastal areas with sparse population over the plain regions. The Central Plains reach over an area of 500 km from west to east and are characterized by nutritious soil and sufficient water resources. The Western Plains have scarce precipitation and few river systems (NSW Gov., 2019).

According to the Australian Government Bureau of Meteorology and its annual climate statements, the entire state of NSW has been experiencing abnormally dry periods, as visible in figure 7 with severe rainfall deficiencies and sporadically, intense rainfall events with record rains during the wet period as presented in figure 8. During the intense rainfall events, major flooding events occur.

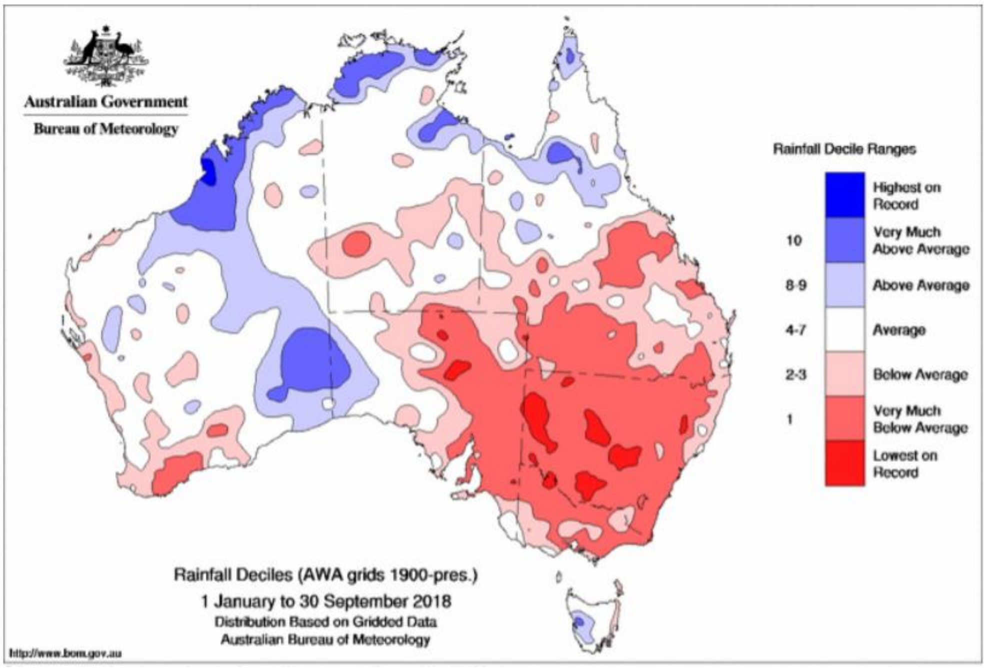


Figure 7:Australian rainfall deciles for the 9 month period from January to September 2018 (Commonwealth of Australia, Bureau of Meteorology, 2018).

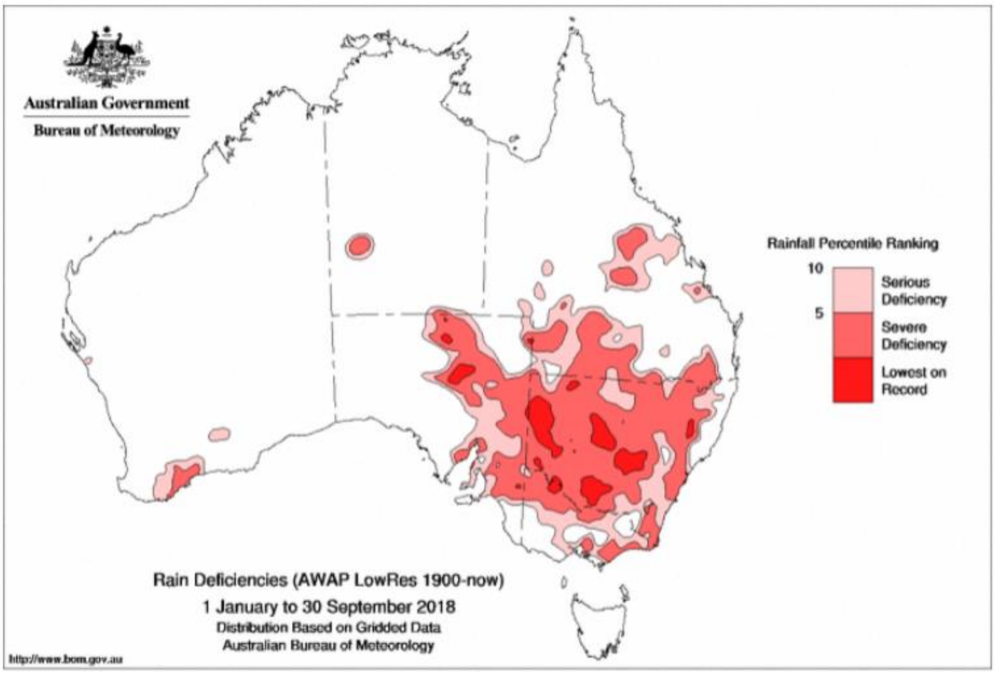


Figure 8: Rainfall deficiencies for the period from January to September 2018 (Commonwealth of Australia, Bureau of Meteorology, 2018).

The catchment of interest is located in the north-east part of the NSW state, right by the coast. Its area is equal to 3.5 km^2 and the creek flowing through it is classified as shallow, having an average depth of 0.1m (NSW Government, Office of Environment & Heritage, 2018). As presented on figure 9, most of the catchment comprises of rural landscape and environmental conservation areas. The urbanized extent of the catchment is minimal, with mostly low-density residential areas and infrastructure such as a highway and a railway track cutting through it. The culvert studied is located under the railway tracks, towards the mouth of the river.

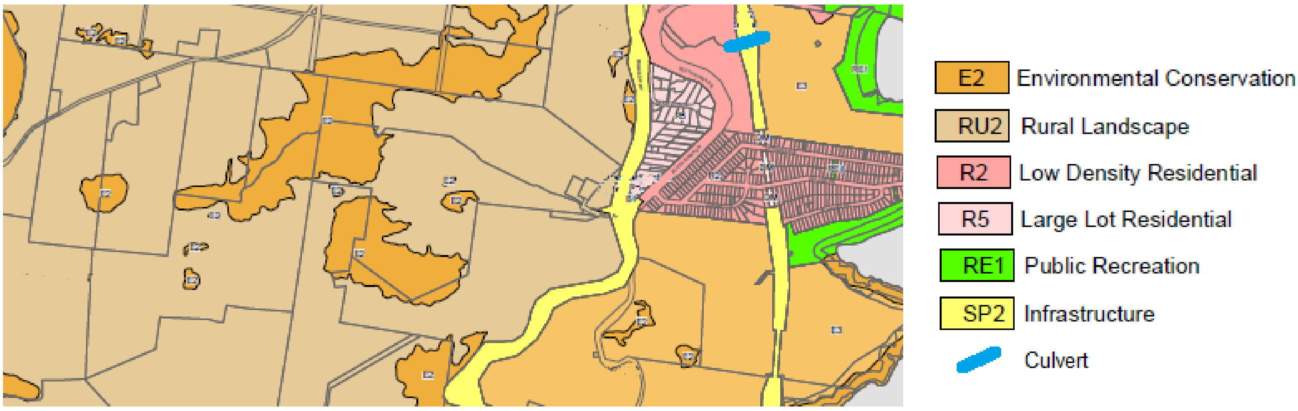


Figure 9: Catchments Local Environmental Plan (2011 to date) (New South Wales Government, 2011).

The entire catchment is divided into fifteen sub-catchments, whose areas vary from 8.0 ha to 44ha. The culvert is located at the downstream part of the fourteenth sub-catchment and the fifteenth sub-catchment is the river's mouth. The exact sub-catchment division has not been, however, provided.

3.2.2. Structure design

The catchment used for this study is a natural catchment, with little modification done to its channel. Towards the mouth of a stream, there is a culvert located below a railway tack. The culvert is constructed of an inlet made of a concrete headwall with widely splayed wing walls and a wide apron. The Underline Crossing (ULX) transitions from a concrete box culvert that is approximately 6.0m long, 2.90m wide and 2.70m high to a concrete arch culvert approximately 42.4m long in a horse shoe profile with mid span of 3.0m. The outlet is constructed of a concrete headwall with wing walls and a "V" shaped invert. The Underline Crossing outlets to a pond.

3.2.3. Storm design

The design storm for the model is calculated based on the procedures listed in the Australian Rainfall and Runoff – method following the 1987 Australian Rainfall and Runoff Procedure according to Pilgrim 1987. In accordance with the 1987 standards, for a small catchment (less than 3 km long), intensity frequency duration coefficients (IFDs) are applied to their centroids, hence the parameters used for the storm design are taken from two stations within the catchment. The coefficients and rainfall intensities for different durations and return periods are taken from the Australian Government Bureau of Meteorology. The IFD coefficients used for the catchment are presented in the table below.

Table 2: Arguments used in storm design.

Argument	Value
Name of gauge	-
Zone	1
Map name	-
Easting	0.
Northing	0.
Elevation [m]	4
2 year 1 hour intensity [mm/h]	4
2 year 12 hour intensity [mm/h]	9.
2 year 72 hour intensity [mm/h]	2.
50 year 1 hour intensity [mm/h]	9
50 year 12 hour intensity [mm/h]	2
50 year 72 hour intensity [mm/h]	6.
F2 - Frequency factor (from ARR)	4.
F50 - Frequency factor (from ARR)	1
Skewness coefficient (G)	0.
Average annual rainfall [mm]	1
Roughness percentage [%]	1
Moisture adjustment factor	0.

3.3. Model set-up

3.3.1. Data

The data provided for the study consists of a set-up hydrologic model (WBNM), which has been selected based on the given location of the catchment evaluated. The prepared WBNM does not take into consideration impervious areas or losses associated with them. Storm events are taken from two gages within the catchment and interpolated for its centroid, as the catchment has a relatively small area. The model contains an outlet structure; the losses are calculated according to the initial loss-constant loss rate.

3.3.2. Data drawbacks

- There is no recorded data to which the model outputs may be compared. All analysis is therefore conducted in reference to the initial WBNM provided, which has been verified as a model generating correct results.
- The model is set up in a very conservative way, assuming the most severe case scenarios.
- The distribution of the sub-catchments has not been provided along with the model. However, that is not an issue, as WBNM is a lumped model and has no spatial dependency. However, the sub-catchment distribution could give insight into the hydrologic process if provided.

3.3.3. Model set-up

The model has been set up in the following manner.

1. Firstly, minor system data gaps have been filled in such as missing inverts, diameters, ground elevations.
2. Validation and correction of any minor system network inconsistencies.
3. Complete the major system network connectivity by placing weirs, correcting flow paths, revising sag locations etc.
4. Assigning major system geometry.
5. Incorporating ditches into the network.
6. Head discharge curves were assigned to gully nodes.
7. Image processing of aerial photos to get understanding of the topography.
8. Sub-catchment delineation from a DEM.
9. Add sub-catchment parameters such as imperviousness, losses etc.
10. Set up channel reaches in tributaries to creek.
11. Running and validating the model.

3.4. Model parameters

WBNM is constructed in a way which allows for great simplification of converting rainfall to runoff, yet still producing high quality results. The parameters in WBNM allow to adjust the model to best fit reality. Overall, WBNM has three main parameters used for flood routing.

- The **Lag Parameter** is used in the conversion of rainfall to runoff, both from pervious and impervious surfaces as well as flood routing and. The parameter depends on the sub-catchment area, where greater sub-catchments will have larger lag time values. WBNM has built in processes which calculate lag time appropriately for each process, hence only one value is required for the entire catchment. Lag time for each sub-catchment is calculated as a product of the area and Lag Parameter. This allows for the application of the Lag Parameter to a vast number of different sub-catchment areas. For natural catchments and pervious areas of urban catchments, the Lag Parameter is assumed to be around 1.6. Setting this parameter to zero results in no routing of runoff from pervious / impervious surfaces. Produced hydrographs will be equal to their corresponding excess rainfall hyetographs. No routing in the stream channel will occur – the hydrograph at the bottom of the channel will be the same as at the top.
- The **Impervious Lag Factor** is used to reduce the Lag Parameter accordingly for the impervious areas covering the sub-catchment. The parameter value is approximately 0.10. If set to zero, no routing of excess rainfall over impervious surfaces will occur. Runoff hydrograph generated from impervious surfaces will be equal to the excess rainfall hyetograph.
- The **Stream Lag Factor** is a parameter used to reduce the Lag Parameter value according to the pervious changes within the stream channel. For unmodified streams, a Stream Lag Factor of 1.0 is used. For modified streams (cleared, straightened, lined) increased velocities will lead to a decrease in lag time. For streams with increased velocity twice, will have a Stream Lag Factor of 0.5. If set to zero, lag time in the stream will be equal to zero. Hydrograph at the bottom of the channel will be the same as at the top of the channel.

Overall, flow routed through a sub-catchment is obtained as follows, a Lag Parameter applies to all the sub-catchments with a Stream Lag Factor applying to the stream in the given sub-catchment and the impervious surfaces lag time is modified based on the Impervious Lag Factor. It is recommended to use a single value of the Lag Parameter for all the sub-catchments and changes to the stream may be done by modifying the Stream lag Factor and the Impervious Lag Factor, however these are also recommended to remain as provided.

Other specifications within the model are as follows:

- Area of sub-catchment – if set to zero local hydrographs from pervious and impervious surfaces will not be generated. The hydrograph at the bottom of the stream channel will be the same as the one on top of the stream channel.

- Baseflow Hydrographs – Since WBNM is an event model, it calculates surface runoff hydrographs based on storm events hence it does not model baseflow. If one wants to introduce a baseflow hydrograph into the output results, this may be done by importing it into the model. When adding the baseflow to WBNM the output results will include runoff volume and depths with the additional flow from the base flow. The runoff volume will then exceed excess rainfall volumes and depths.
- Calibration using recorded rainfall and flood data – In order to ensure the model is optimal it is always recommended to calibrate WBNM using recorded flood data when possible. If full hydrograph data is unavailable, it is possible to use maximum water levels to check the flood peak discharges.
- Design Storms – Design storms are calculated based on the procedures given in the Australian Rainfall and Runoff. The durations of these storms are from 5 minutes to 72 hours and their average recurrence intervals (ARI) range from 1 to 500 years. Based on the nine basic IFD coefficients provided, WBNM calculates the average rainfall intensity which is then distributed according to an appropriate design storm temporal pattern.
- Discharge through culverts and weirs - a discharge factor equal to zero models a completely blocked outlet. (0.9 = 90% of full discharge through the whole structure).
- Embedded design storms – embedded design storms allow the simulation of partially or completely full streams and storage reservoirs. With this feature, it is possible to specify an average recurrence interval (ARI) and the duration of a critical burst and the ARI along with the duration of a longer storm event within which the burst occurs.
- Flood Routing in Storage Reservoirs / Flood Detention Basins – Outlet structures and local structures follow mainly the same flood flow routing procedures. Inflows into the storage reservoir are routed based on a level pool routing procedure. This requires a table of elevation-storage volume-discharge values (HSQ). The elevation-storage and elevation-discharge values use a linear interpolation between the elevations specified.
- Probable Maximum Precipitation – PMP are calculated in the program according to the Australian Bureau of Meteorology Bulletin 53 (1994), which may be applied to catchments with areas up to 1000km^2 and storm durations up to 6 hours. Based on this approach, average rainfall intensity is broadcasted into the Bureau's generalised temporal pattern. Although Bulletin 53 allows for spatial variation of PMP, WBNM does not calculate it.
- Storage volumes in outlet structures and local structures – if set to zero, no storage routing will occur. The outflow hydrograph will be the same as the inflow hydrograph.
- Time Step - In most cases the time step used for routing flow will be equal to that one of the rainfall hyetographs. In some cases, it is necessary for the time step to be smaller – if the lag time of a sub-catchment becomes smaller than the calculation time step, the nonlinear routing equation will be no longer possible. This is common with large discharges

over small sub-catchments. In some cases, when the sub-catchment area is very small, in order to avoid using a very small-time step, it is recommended to replace nonlinear stream channel routing by a time delay. Local or outlet structures may also require a small calculation time step.

4. Sensitivity Analysis based on parameter evaluation

4.1. Approach

In order to analyse the sensitivity of the model to changes in the impervious area cover (urbanization), a simple study based on parameter evaluation has been conducted. Having an initially set up model, the parameter values of imperviousness have been modified and the model was ran according to them. Twelve runs were done, each with different values of impervious cover: 1%, 2%, 3%, 4%, 5%, 7%, 10%, 15% 20%, 25%, 33% and 50%. The impervious cover was distributed evenly across all the sub-catchments. Imperviousness was the only parameter manipulated in the runs. The flow at the outlet of the culvert structure placed in sub-catchment 14 was used to evaluate the difference in the results, as this was the point of reference chosen by the client. The sensitivity analysis was conducted in reference to the initial values produced by the WBNM provided, with no impervious cover (treated as the base) and not actual recorded data (unavailable). Hence, the analysis evaluates how the simulated results differ from the initial results of the model, which has been confirmed by the client as accurate.

The choice of the impervious area cover (1%, 2%, 3%, 4%, 5%, 7%, 10%, 15% 20%, 25%, 33% and 50%) have been selected based on the required intervals for the analysis and the terrain in its current state – with minimal cover of impervious surfaces of approximately 10%. The maximum has been chosen as 50% since, majority of the catchment is either an environmental conservation or rural landscapes, which are unlikely to be all covered with impervious surfaces. The values of 7% and 33% impervious area coverage have been chosen in the second part of the analysis – the Two Point Method.

In order to estimate the effect changing impervious area percentage has on the model, the following performance evaluation coefficients will be investigated:

- error index coefficients (Root Mean Square Error, Percentage Bias),

- standard regression coefficient (Pearson's correlation coefficient (r)),

- dimensionless evaluation parameter (Nash-Sutcliffe efficiency).

The results will be generated for the standard annual recurrence intervals (ARI) stated in the ARR, to see if the intensity of the storm will have any influence on the model's sensitivity to impervious cover. The most commonly checked ARI's are the twenty, fifty and one hundred-year ARI.

4.2. Parameter Evaluation

As stated previously, four evaluation coefficients will be used to conduct the analysis of how sensitive the model is to changes in the impervious cover of the catchment: Root Mean Square Error, Percentage Bias, Pearson's correlation coefficient and the dimensionless evaluation parameter, Nash-Sutcliffe efficiency. The model evaluation coefficients used for the analysis are briefly described below.

The **Root Mean Square Error (RMSE)** is the standard deviation of residuals (prediction errors). Residuals are a measure of how far data points are from the reference values and RMSE is a measure of how spread the residuals are. The smaller the RMSE error, the smaller the error and the closer the estimation to the compared base values.

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n}\left(v_i^{obs} - v_i^{sim}\right)^2}{n}}$$

$$coefficient \geq 0$$

Where: v_i^{obs} – observed values or values used as the reference point
v_i^{sim} – values obtained from the model simulations
n – number of values

The **Percent Bias (PBIAS)** is a quantitative term describing the difference between the average of the results generated and the observed / reference value. By Jensen's inequality, a convex function as transformation will introduce positive bias, while concave functions result in negative bias.

$$PBIAS = \frac{100 \cdot \sum_{i=1}^{n}\left(v_i^{obs} - v_i^{sim}\right)}{\sum_{i=1}^{n} v_i^{obs}}$$

Where:
v_i^{obs} – observed values or values used as the reference point
v_i^{sim} – values obtained from the model simulations

Pearson correlation coefficient (r) is a measure of correlation between two variables. It varies from +1 to -1, where +1 indicates absolute positive correlation, zero indicates no correlation and -1 indicates negative correlation.

$$r = \frac{\sum_{i=1}^{n}\left(v_i^{obs} - v_{mean}^{obs}\right)\left(v_i^{sim} - v_{mean}^{sim}\right)}{\sqrt{\sum_{i=1}^{n}(v_i^{obs} - v_{mean}^{obs})^2}\sqrt{\sum_{i=1}^{n}\left(v_i^{sim} - v_{mean}^{sim}\right)^2}}$$

$$coefficient -1 \leq r \leq 1$$

Where:
v_i^{obs} – observed values or values used as the reference point
v_{mean}^{obs} – mean observed value or value used as the reference point
v_i^{sim} – values obtained from the model simulations
v_{mean}^{sim} – mean value obtained from the model simulations

Nash-Sutcliffe efficiency (NSE) parameter ranges from negative infinity to one.

- NSE = 1 indicates a perfect match of the modelled discharge to the reference data,
- NSE = 0 indicates that the predictions are as accurate as the mean of the observed data
- NSE < 0 indicates that the observed/reference mean is a better predictor than the model

Overall, the closer the model efficiency is to 1, the more accurate the model is and the results generated by it are closer to the reference values.

$$NSE = 1 - \frac{\sum_{i=1}^{n}\left(v_i^{obs} - v_i^{sim}\right)^2}{\sum_{i=1}^{n}(v_i^{obs} - v_{mean}^{obs})^2}$$

$$coefficient\ 0 \leq NSE \leq 1$$

Where:
v_i^{obs} – observed values or values used as the reference point
v_{mean}^{obs} – mean observed value or value used as the reference point
v_i^{sim} – values obtained from the model simulations

4.3. Results

4.3.1. Peak flow Analysis

Table 3 presents the results of the peak flow generated for all the storm events (ARI: 1, 2, 5, 10, 20, 50 and 100), with different impervious area catchment covers, ranging from 1% to 50%. The peak flow values used for the analysis are from the outlet of the culvert, as required in the initial project analysis. Figure 10 presents the peak flow values compared on a graph amongst themselves. It is visible that although the peak flow values differ from each other, they all maintain a constant trend.

Table 4 presents a difference between the impervious cover scenarios compared to the base peak flow at the outlet. By comparing the results from table 3 to the base values, the increase in peak flow at the outlet compared to the reference scenario (0% imperviousness) is approximately from 0.05 to 0.629 m^3/s for the less severe case scenarios (impervious area covers up to 10%). According to table 4, impervious cover of 50% of the catchment area may result in the increase of peak flow at the outlet by up to 3.83 m^3/s. However, to estimate the effect of these changes, it is necessary to conduct a hydraulic analysis.

Table 3: Peak flow values for different storm intensities and different impervious area fractions.

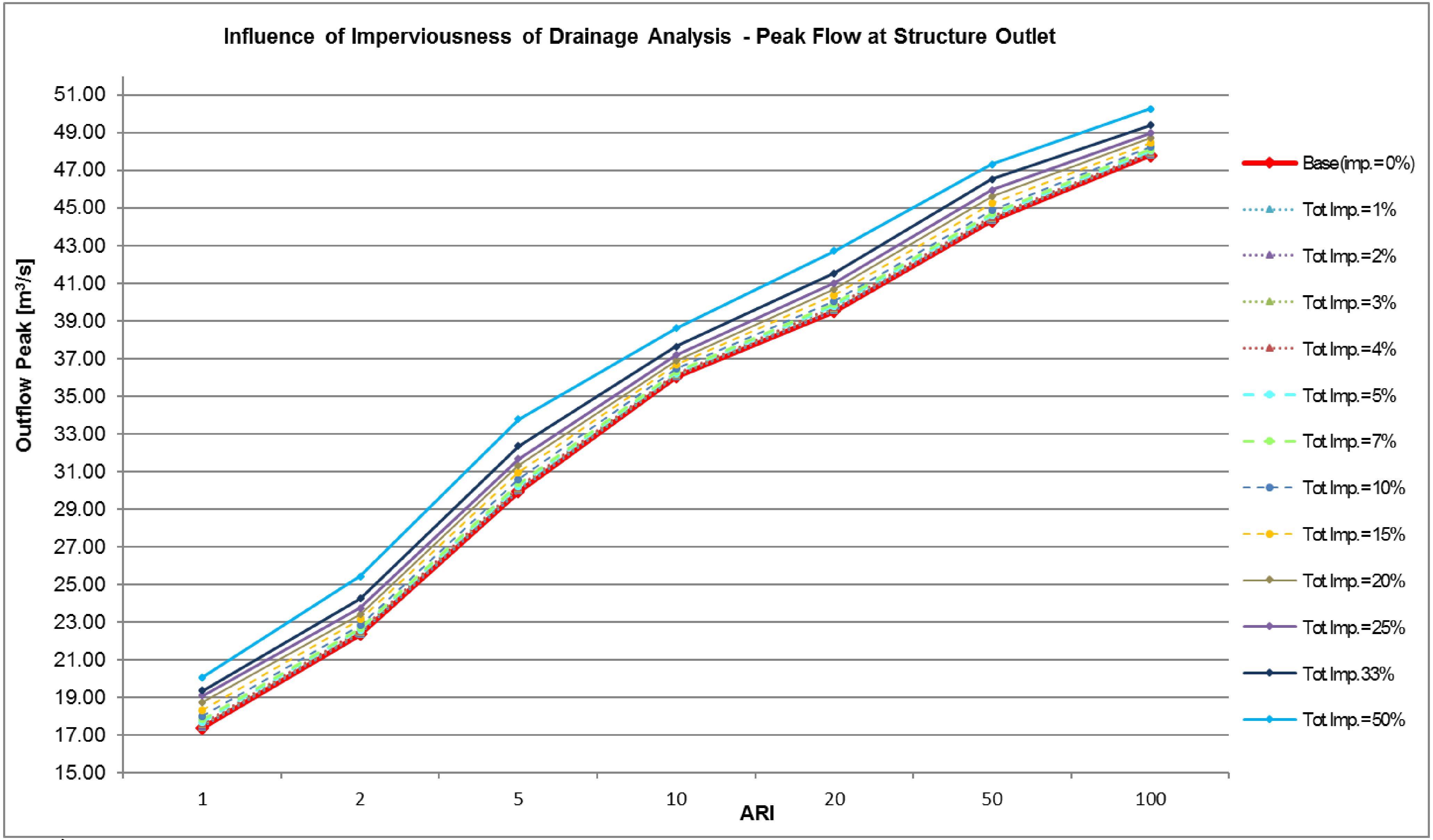
	Total Impervious Area [%]	**ARI [years]**						
		1	**2**	**5**	**10**	**20**	**50**	**100**
Outflow Peak [m3/s]	Base (0% imp.)	17.38	22.33	29.92	36.03	39.51	44.31	47.79
	1	17.44	22.38	29.98	36.07	39.56	44.36	47.84
	2	17.49	22.43	30.05	36.10	39.62	44.41	47.88
	3	17.55	22.48	30.11	36.14	39.67	44.47	47.92
	4	17.62	22.53	30.17	36.18	39.73	44.53	47.97
	5	17.69	22.57	30.23	36.22	39.78	44.59	48.01
	7	17.82	22.68	30.34	36.30	39.89	44.72	48.10
	10	18.01	22.84	30.55	36.44	40.05	44.91	48.24
	15	18.33	23.13	30.93	36.67	40.34	45.26	48.46
	20	18.71	23.41	31.29	36.90	40.67	45.60	48.70
	25	19.07	23.73	31.67	37.18	41.00	45.96	48.96
	33	19.35	24.25	32.34	37.63	41.53	46.52	49.38
	50	20.07	25.42	33.75	38.61	42.70	47.32	50.26

Table 4: Difference between reference (base) peak flow values and the simulated peak flow at the outlet.

	Total Impervious Area [%]	**ARI [years]**						
		1	**2**	**5**	**10**	**20**	**50**	**100**
Outflow Peak [m3/s]	Base (0% imp.)	17.38	22.33	29.92	36.03	39.51	44.31	47.79
	1	0.05	0.05	0.06	0.04	0.05	0.05	0.04
	2	0.10	0.10	0.12	0.08	0.11	0.11	0.08
	3	0.17	0.14	0.18	0.12	0.16	0.16	0.13
	4	0.24	0.19	0.24	0.15	0.22	0.22	0.17
	5	0.30	0.24	0.30	0.19	0.27	0.29	0.22
	7	0.43	0.34	0.42	0.28	0.38	0.41	0.31
	10	0.63	0.50	0.63	0.41	0.54	0.60	0.44
	15	0.94	0.80	1.00	0.65	0.83	0.95	0.67
	20	1.32	1.08	1.37	0.88	1.16	1.29	0.90
	25	1.69	1.40	1.75	1.16	1.49	1.65	1.17
	33	1.97	1.92	2.42	1.61	2.02	2.21	1.59
	50	2.69	3.09	3.83	2.59	3.19	3.01	2.46

In the most severe cases, the peak flow is increased from 39.51 m^3/s (reference peak flow) to 42.70 m^3/s (50% imperviousness peak flow) for 20-year ARI. For 50-year ARI, the peak flow increases from 44.30 m^3/s (reference peak flow) to 47.32 m^3/s (50% imperviousness peak flow). For 100-year ARI, the peak flow increases from 47.79 m^3/s (reference peak flow) to 50.26 m^3/s (50% imperviousness peak flow). As presented in table 5, impervious cover up to 15% has a very miniscule impact on increasing the peak flow.

Based on the table above (Table 4) additional hydraulic analysis should be conducted to properly evaluate the influence of increasing the peak flow, for each reported increment.



4

Figure 10: Peak flow for different storm intensities and different impervious area extents.

4.3.2. Hydrograph Analysis

Figure 11 presents the hydrographs generated for a 1-year ARI for the impervious area cover of the base (0%) and 5%, 7%, 20%, 25%, 33% and 50%. The general shape of the hydrograph remains the same for most of the cases, however increasing the catchments imperviousness by 50% increases the peak the most significantly, with decreasing its time of arrival and extending the time of the crest segment duration. Time of arrival to peak for increasing impervious cover decreases up to twenty minutes compared to the base flow, however, since the output of this model is generated every 5 minutes, it is impossible to get the exact value of when the peak flow arrives for the simulated hydrograph.

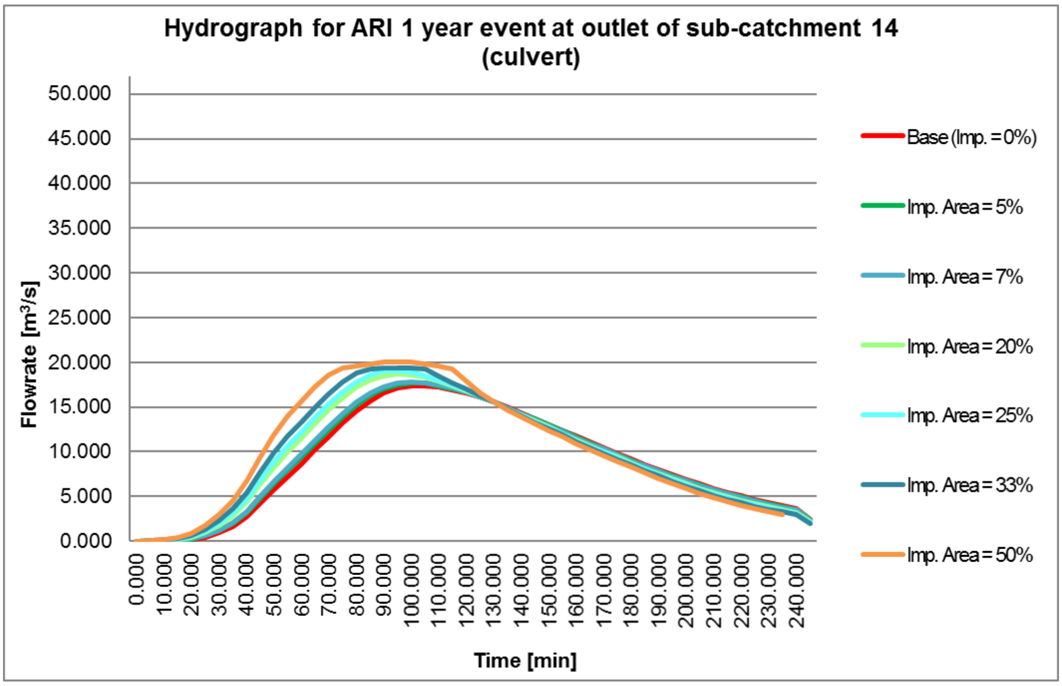


Figure 11: Comparison of hydrographs at the outlet of the culvert, generated with different impervious area extents for a 1-year ARI flood.

Figure 12 presents the hydrograph generated at the outflow from the structure in sub-catchment 14, taking into consideration 5%, 7%, 20%, 25%, 33% and 50% impervious cover of the catchment for a 2-year ARI flood. The general shape of the hydrograph is maintained for all the cases. The 7% impervious cover hydrograph is almost identical to that of no impervious cover. Time of arrival to peak for increasing impervious cover decreases up to fifteen minutes compared to the base flow.

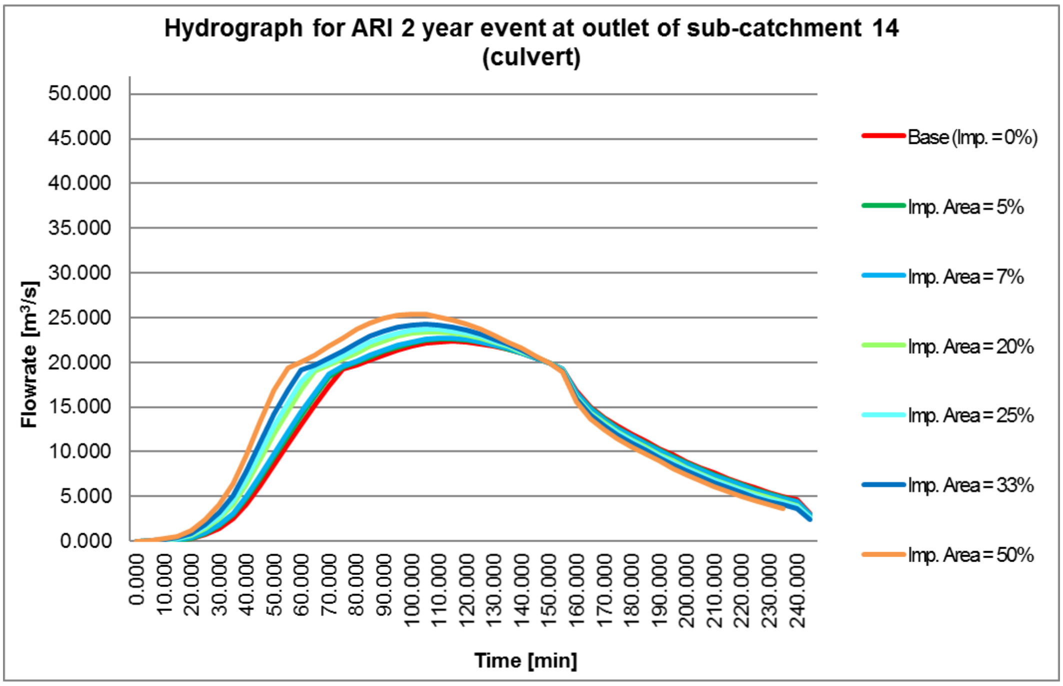


Figure 12: Comparison of hydrographs at the outlet of the culvert, generated with different impervious area extents for a 2-year ARI flood.

Figure 13 presents the hydrograph generated at the outflow from the structure in sub-catchment 14, taking into consideration 5%, 7%, 20%, 25%, 33% and 50% impervious cover of the catchment for a 5-year ARI flood. The general shape of the hydrograph is maintained for all the cases. Time of arrival to peak for increasing impervious cover decreases up to twenty minutes compared to the base flow.

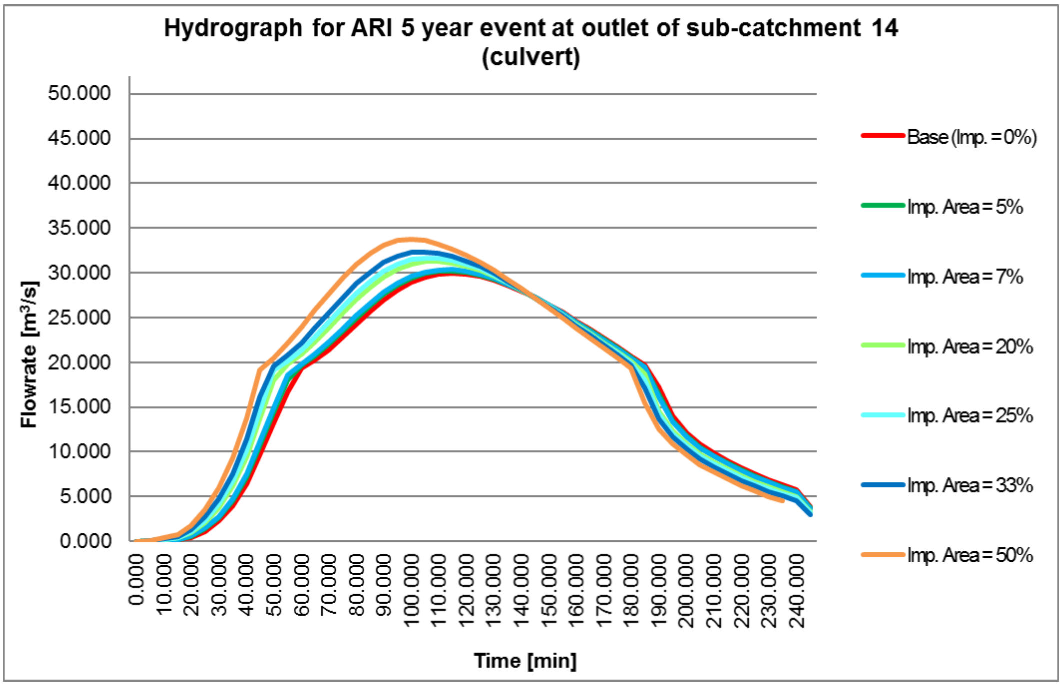


Figure 13: Comparison of hydrographs at the outlet of the culvert, generated with different impervious area extents for a 5-year ARI flood.

Figure 14 presents the hydrograph generated at the outflow from the structure in sub-catchment 14, taking into consideration 5%, 7%, 20%, 25%, 33% and 50% impervious cover of the catchment for a 100-year ARI flood. The general shape of the hydrograph is maintained for all the cases. Time to peak in the case of an increased impervious cover decreases up to ten minutes compared to the base flow and the difference in intensity of the peak flow for the most severe case is up 2.58 m^3/s.

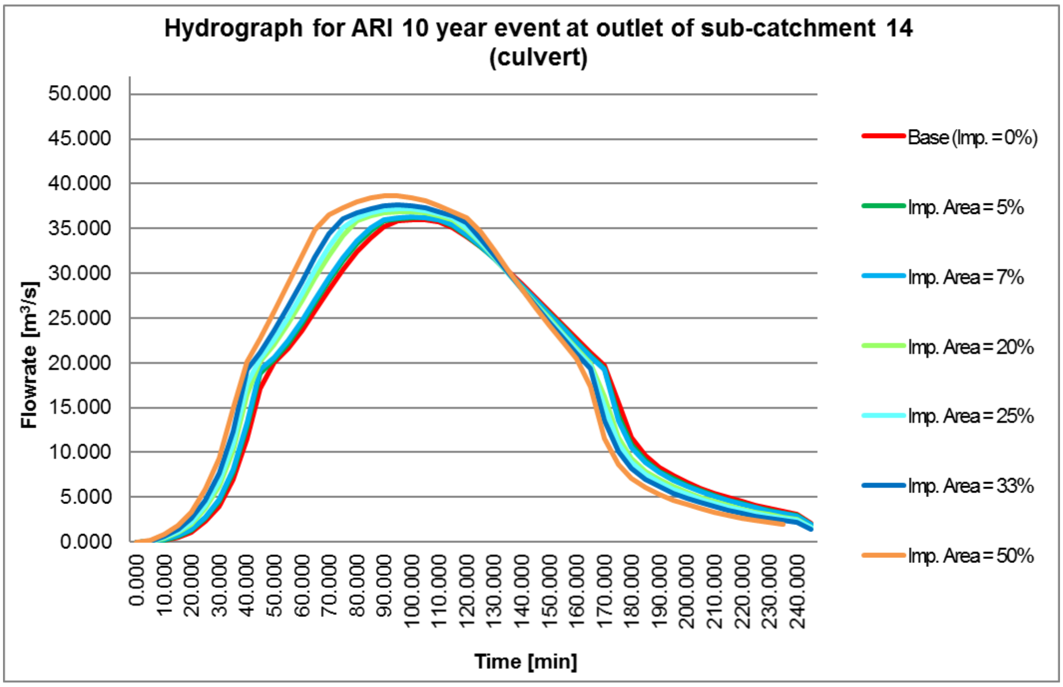


Figure 14: Comparison of hydrographs at the outlet of the culvert, generated with different impervious area extents for a 10-year ARI flood.

Figure 15 presents the hydrograph generated at the outflow from the structure in sub-catchment 14, taking into consideration 5%, 7%, 20%, 25%, 33% and 50% impervious cover of the catchment for a 20-year ARI flood. The general shape of the hydrograph is maintained for all the cases. Time to peak in the case of an increased impervious cover decreases up to ten minutes compared to the base flow and the difference in intensity of the peak flow for the most severe case is up to 3.19 m^3/s.

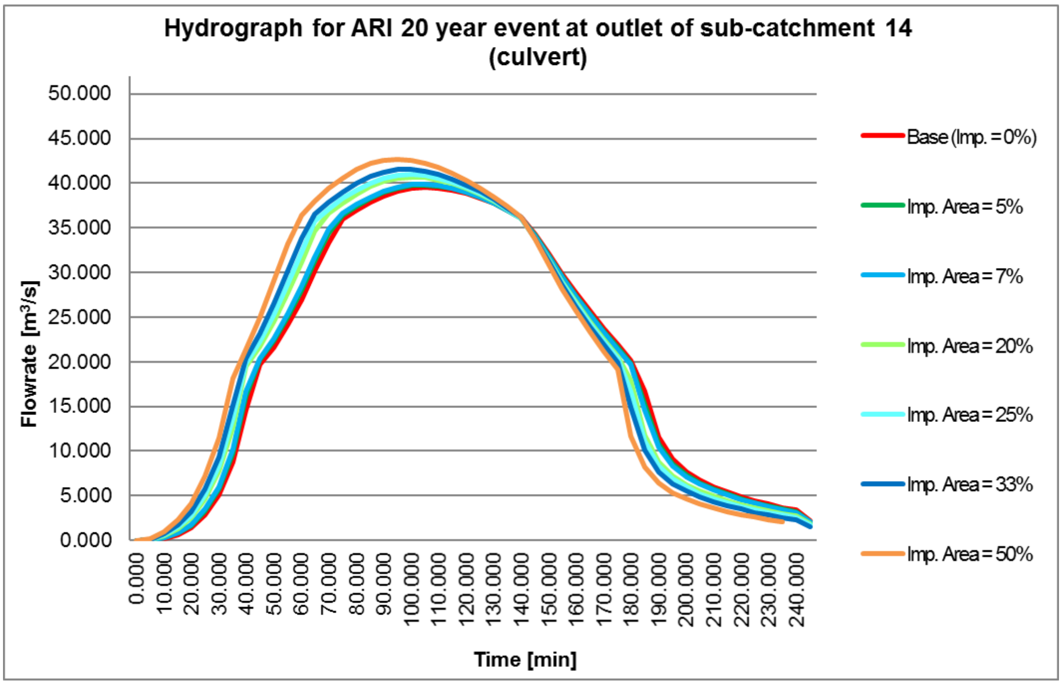


Figure 15: Comparison of hydrographs at the outlet of the culvert, generated with different impervious area extents for a 20-year ARI flood.

Figure 16 presents the hydrograph generated at the outflow from the structure in sub-catchment 14, taking into consideration 5%, 7%, 20%, 25%, 33% and 50% impervious cover of the catchment for a 50-year ARI flood. Based on the graph, it is visible that the hydrograph generated with imperviousness equal to 50% has an increased flow rate, with the peak flow arriving approximately 10 minutes before the reference peak flow. The difference in intensity of the peak flow for the most severe case is up to 3.01 m^3/s.

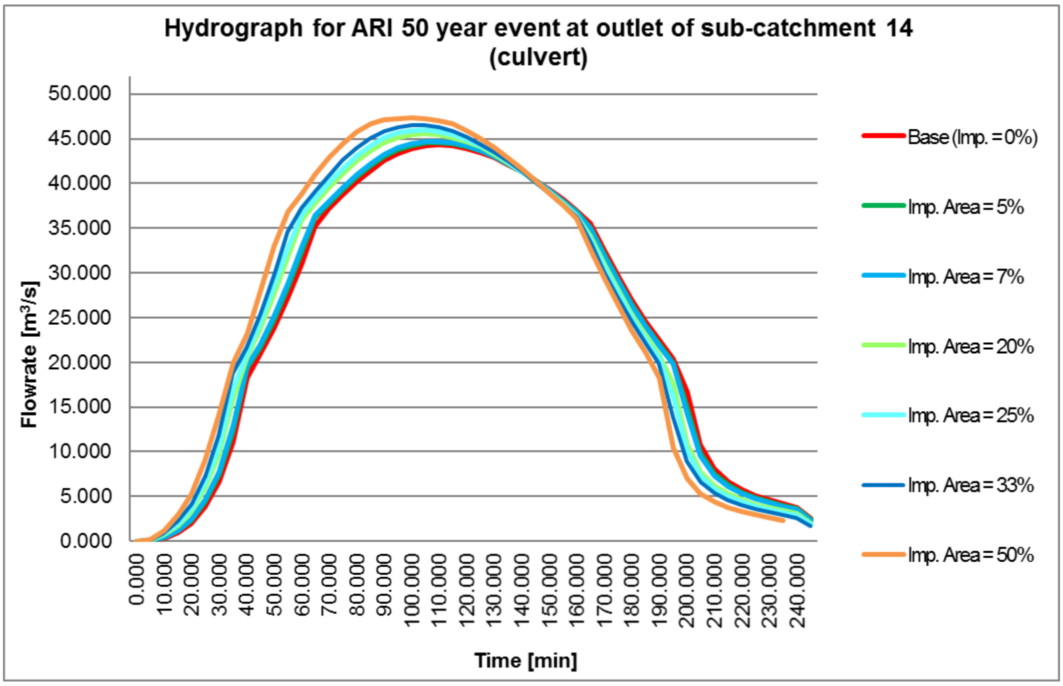


Figure 16: Comparison of hydrographs at the outlet of the culvert, generated with different impervious area extents for a 50-year ARI flood.

Figure 17 presents the hydrograph generated at the outflow from the structure in sub-catchment 14, taking into consideration 5%, 25% and 50% impervious cover of the catchment for ARI of 100 years. Based on the graph, it is visible that the hydrograph generated with imperviousness equal to 50% has an increased flow rate and time to peak slightly less than 10 minutes before the reference peak flow. Since the output of this model is generated every 5 minutes, it is impossible to get the exact value of when the peak flow arrives for the simulated hydrograph. However, from visual analysis it appears that the peak flow arrives slightly later than that of simulated values for a 50-year ARI flood. The difference in intensity of the peak flow for the most severe case is up to 2.46 m^3/s.

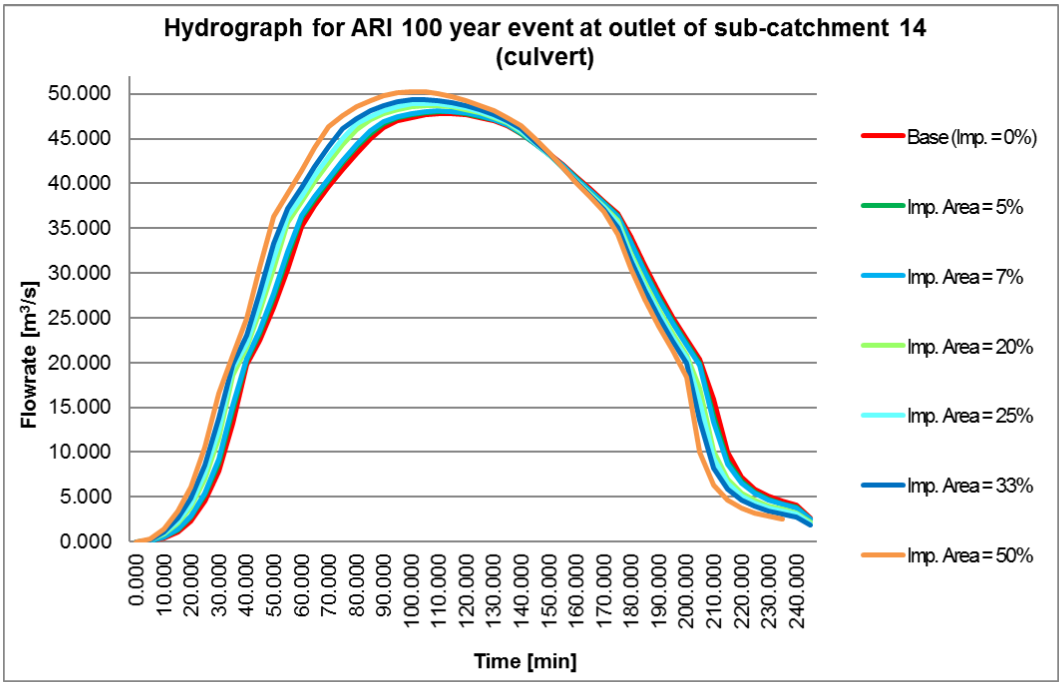


Figure 17: Comparison of hydrographs at the outlet of the culvert, generated with different impervious area extents for a 100-year ARI flood.

4.3.3. Model evaluation coefficients

Tables 5 to 11 present the values of the model evaluation coefficients obtained, based on analysing the peak flow values generated by the model, for all of the ARI.

Table 5: Sensitivity analysis parameter summary for 1-year ARI.

Model Evaluation Coefficients:	Impervious Cover											
	1%	2%	3%	4%	5%	7%	10%	15%	20%	25%	33%	50%
RMSE	0.07	0.13	0.20	0.27	0.33	0.47	0.66	0.98	1.29	1.60	2.07	3.00
PBIAS [%]	-0.31	-0.62	-0.92	-1.23	-1.53	-2.14	-3.04	-4.54	-6.02	-7.48	-9.78	-14.84
Pearson's correlation coefficient	1.000	1.000	1.000	0.999	0.999	0.997	0.995	0.989	0.982	0.973	0.957	0.919
NSE	1.000	0.999	0.999	0.998	0.997	0.993	0.987	0.971	0.949	0.921	0.869	0.735

Table 6: Sensitivity analysis parameter summary for 2-year ARI

Model Evaluation Coefficients:	Impervious Cover											
	1%	**2%**	**3%**	**4%**	**5%**	**7%**	**10%**	**15%**	**20%**	**25%**	**33%**	**50%**
RMSE	0.08	0.15	0.23	0.31	0.38	0.53	0.75	1.09	1.44	1.75	2.25	3.24
PBIAS [%]	-0.24	-0.49	-0.74	-0.98	-1.23	-1.71	-2.43	-3.62	-4.81	-5.96	-7.79	-11.86
Pearson's correlation coefficient	1.000	1.000	1.000	0.999	0.999	0.998	0.996	0.992	0.987	0.982	0.971	0.946
NSE	1.000	1.000	0.999	0.998	0.998	0.995	0.991	0.980	0.966	0.949	0.916	0.833

Table 7: Sensitivity analysis parameter summary for 5-year ARI.

Model Evaluation Coefficients:	Impervious Cover											
	1%	**2%**	**3%**	**4%**	**5%**	**7%**	**10%**	**15%**	**20%**	**25%**	**33%**	**50%**
RMSE	0.09	0.19	0.28	0.37	0.46	0.64	0.90	1.31	1.71	2.10	2.67	3.89
PBIAS [%]	-0.19	-0.39	-0.58	-0.77	-0.96	-1.34	-1.90	-2.83	-3.75	-4.65	-6.06	-9.28
Pearson's correlation coefficient	1.000	1.000	1.000	0.999	0.999	0.998	0.997	0.994	0.989	0.984	0.976	0.952
NSE	1.000	1.000	0.999	0.999	0.998	0.996	0.992	0.984	0.973	0.959	0.933	0.863

Table 8: Sensitivity analysis parameter summary for 10-year ARI.

Model Evaluation Coefficients:	Impervious Cover											
	1%	**2%**	**3%**	**4%**	**5%**	**7%**	**10%**	**15%**	**20%**	**25%**	**33%**	**50%**
RMSE	0.11	0.23	0.34	0.45	0.56	0.76	1.05	1.51	1.96	2.39	3.05	4.28
PBIAS [%]	-0.13	-0.26	-0.39	-0.52	-0.64	-0.89	-1.27	-1.90	-2.50	-3.10	-4.06	-6.24
Pearson's correlation coefficient	1.000	1.000	1.000	0.999	0.999	0.998	0.997	0.994	0.991	0.986	0.979	0.961
NSE	1.000	1.000	0.999	0.999	0.998	0.996	0.993	0.986	0.976	0.965	0.943	0.893

Table 9: Sensitivity analysis parameter summary for 20-year ARI.

Model Evaluation Coefficients:	**Impervious Cover**											
	1%	**2%**	**3%**	**4%**	**5%**	**7%**	**10%**	**15%**	**20%**	**25%**	**33%**	**50%**
RMSE	0.11	0.22	0.34	0.45	0.56	0.77	1.09	1.59	2.01	2.44	3.07	4.40
PBIAS [%]	-0.11	-0.22	-0.33	-0.44	-0.55	-0.77	-1.10	-1.64	-2.17	-2.70	-3.51	-5.43
Pearson's correlation coefficient	1.000	1.000	1.000	1.000	0.999	0.999	0.998	0.995	0.992	0.989	0.983	0.966
NSE	1.000	1.000	0.999	0.999	0.999	0.997	0.994	0.988	0.981	0.972	0.956	0.912

Table 10: Sensitivity analysis parameter summary for 50-year ARI.

Model Evaluation Coefficients:	**Impervious Cover**											
	1%	**2%**	**3%**	**4%**	**5%**	**7%**	**10%**	**15%**	**20%**	**25%**	**33%**	**50%**
RMSE	0.13	0.25	0.38	0.49	0.60	0.81	1.14	1.68	2.16	2.62	3.35	4.70
PBIAS [%]	-0.10	-0.20	-0.29	-0.39	-0.49	-0.68	-0.96	-1.43	-1.88	-2.35	-3.05	-4.71
Pearson's correlation coefficient	1.000	1.000	1.000	1.000	0.999	0.999	0.998	0.995	0.992	0.989	0.983	0.967
NSE	1.000	1.000	0.999	0.999	0.999	0.997	0.995	0.989	0.982	0.974	0.957	0.919

Table 11: Sensitivity analysis parameter summary for 100-year ARI.

Model Evaluation Coefficients:	**Impervious Cover**											
	1%	**2%**	**3%**	**4%**	**5%**	**7%**	**10%**	**15%**	**20%**	**25%**	**33%**	**50%**
RMSE	0.13	0.25	0.38	0.49	0.61	0.84	1.17	1.71	2.20	2.65	3.32	4.68
PBIAS [%]	-0.09	-0.18	-0.26	-0.35	-0.43	-0.60	-0.86	-1.28	-1.71	-2.11	-2.75	-4.26
Pearson's correlation coefficient	1.000	1.000	1.000	1.000	0.999	0.999	0.998	0.996	0.993	0.990	0.985	0.969
NSE	1.000	1.000	1.000	0.999	0.999	0.998	0.995	0.990	0.984	0.977	0.963	0.930

4.4. Discussion

Based on the conducted analysis along with some basic statistical calculations, it appears that the model is less sensitive to the parameter of imperviousness with increasing storm intensity. This is supported by the values of the model evaluation coefficients from tables 8, 9, 10 and 11, where for the higher intensity storm, the values represent a better fit of the simulated data to the base.

For all the storm events, starting from 1- year ARI up to 100-year ARI the Root Mean Square Error increases with increasing imperviousness. As the overall values of the coefficient range from 0.07 to 1.51 for the small storm intensity (1 to 10-year ARI) for physically reasonable impervious area coverages (up to 15%). For larger intensity storms (20 to 100-year ARI), the values range from 0.11 to 1.71 for physically reasonable impervious area coverages (up to 15%). For these values it may be assumed that the simulated results compared to the base value are very close. For the most severe cases (33% and 50% imperviousness) high storm intensity, RMSE is equal to 4.40 for 20-year ARI, 4.70 for 50-year ARI and 4.68 for 100-year ARI. 8. This indicated larger variation of the generated results from the reference data, yet still not extremely.

As all the hydrographs presented in figures 11 – 17 display concave functions, the percent bias coefficient obtained is negative. For the 1-year ARI, PBIAS ranges from -0.31 to -14.84 for the increasing imperviousness and for the 5-year ARI, PBIAS ranges from -0.19 to -9.28. For the 10-year ARI, PBIAS ranges from -0.13 to -6.24 for the increasing imperviousness and for the 20-year ARI, PBIAS ranges from -0.11 to -5.43. Finally, for the 50-year ARI, PBIAS ranges from -0.10 to -4.71 for the increasing imperviousness and for the 100-year ARI, PBIAS ranges from -0.09 to -4.26. The values indicate that the simulated data tends to be larger than the reference (base) data, however the difference is inversely proportional to the base, as with increasing storm intensity, the difference decreases.

Pearson's correlation coefficient is either equal to one or nearly equal to one for all the different storm intensity cases for different imperviousness extents, suggesting a high correlation of the simulated data to the base model (0% imperviousness). For the low annual recurrence interval events, Pearson's correlation coefficient ranges from 1.000 to 0.961 (1-10-year ARI) and for high annual recurrence interval events, Pearson's correlation coefficient ranges from 1.000 to 0.969 (20-100-year ARI). As visible in tables 9 to 11, the coefficient remains closer to one with increasing storm intensity.

Finally, according to the results, the Nash-Sutcliffe parameter for the 1-year ARI, ranges from 1.00 to 0.735 for the increasing imperviousness and for the 5-year ARI, it ranges from 1.000 to 0.863. For the 10-year ARI, NSE ranges from 1.000 to 0.893 for the increasing imperviousness and for the 20-year ARI, NSE ranges from 1.000 to 0.912. Finally, for the 50-year ARI, NSE ranges from 1.000 to 0.919 for the increasing imperviousness and for the 100-year ARI, NSE ranges from 1.000 to 0.930. The results indicate that the goodness of fit of the modelled flow to the reference increases with increasing storm intensity. Overall the results generated by the model with lower impervious area cover present a very good fit to the reference data, which is visible in figures 11 to 17.

Overall, the analysis of how the model results vary depending on changes in imperviousness based on model evaluation coefficients indicated, that increasing the storm intensity, decreases the variation between the reference and modelled results. The model is less sensitive to changes to the parameter of imperviousness for higher storm intensities. However, increasing the impervious cover does lead to a decreased time to peak, which is an important factor when thinking of flood protection. To get a more accurate understanding of how increasing imperviousness would affect the catchment in practice, it would be necessary to input the results into a hydraulic model of the catchment. As the aim of this study is to determine the sensitivity of the model to changing the parameter of imperviousness, this will not be further investigated.

4.5. Observations and limitations

There are several observations and limitations to this analysis that must be taken into account in order to get a good understanding of how reliable the results generated are. Firstly, the initial losses for impervious surfaces equal to zero in the model represent very rigorous conditions in the catchment. This has an impact on the results of this analysis. However, in order to approximate the influence losses of impermeable surfaces may have on the results generated by WBNM, another sensitivity analysis should be conducted. Although it is necessary to note, that some losses may occur over the impermeable surfaces, which if these are equal to zero, there isn't any.

Secondly, the data on how the fifteen sub-catchments are localized within catchment has not been provided. It appears that the last sub-catchment (SUB_15) with the area of 31.4ha is located after the culvert structure. Assigning impermeable area to this sub-catchment will have no influence on the results generated for this analysis, as the analysis is done for the flow values at the outlet structure located in sub-catchment 14. Since the area of sub-catchment 15 is most likely not included in the investigated area, the actual total impermeable area used in the study may be slightly smaller than originally assumed.

Figure 18: Distribution of impervious surfaces over the investigated catchment.
Image from google maps.

As presented in figure 18, the distribution of the impervious areas in a uniform manner over the entire catchment does not represent reality. In the case of the catchment, the impervious areas appear to be focused on the downstream part of the catchment, with very few buildings upstream (presented in figure 18). Hence, the case simulated for this analysis is very severe (50% impervious cover), while the actual area covered with impervious surfaces is in the smaller extent (maximum up to 20%) and differently distributed than that assumed in this study. Taking the impervious area distribution into account would have a significant impact on the results, which would be much more realistic. But as the overall aim of the analysis is to study the effect one parameter has on the entire model, the distribution of the impervious areas is not that crucial in the study. However, it is important to remember, that the scenarios considered are the ones most rigorous and conservative.

Additionally, the land use of the catchment comprises mainly of rural landscapes (RU2), environmental conservation areas (E2) (upstream) and of low density (R2) and large lot residential (R5) areas (downstream), as presented on figure 9. The current development of the upper part of the catchment may indicate that not many changes will be done to this area in the upcoming years and hence the severe parameter conditions used, have no reflection in the reality.

Finally, based on the results generated from the model evaluation coefficients analysis, the changes to the model outputs for the physically plausible situations (impervious cover up to 25%), the model results do not change very significantly.

5. Two-point Technique

The second method used to analyse the sensitivity of the model to a change in the parameter of imperviousness is the Two-point Technique, which was proposed for the first time by Rosenblueth in 1973 (Rosenblueth, 1973) and is one of several point estimate methods. The largest advantage of using the Two-point Technique is that it does not require multiple functions but only the average and variation of the data parameters. Chang et a. (1995) has proven, that the accuracy and reliability of point estimate methods, amongst them the Two-point Technique deteriorated with the increase in the number of parameters and degree of model non-linearity used. In the following assessment, it has been assumed that the parameters used for the study are not correlated.

Assume that *y'* is a variable set based on the models functioning and is a function based on *m*-many parameters and variables (p_i) encumbered with certain uncertainties:

$$y' = f(p_1, p_2, \dots, p_m)$$

Since the uncertainty of the parameters p_i (i=1,2,…,m) is treated as a random variable, *y'* is also a random variable. The N^{th} statistical moment for a chosen initial variable estimated as follows:

$$\sum (y')^n = \frac{1}{2^m}[(y'_{++\dots m})^n + (y'_{\pm\dots m})^n + (y'_{--\dots m})^n]$$

(8)

Where:

m – is the number of parameters and variables encumbered with uncertainty,

$y'_{++\dots m}$ – denoted all the permutations of the addition and subtraction on the m positions:

$$y' = f(\overline{p_1} \pm s_{p_1}, \overline{p_2} \pm s_{p_2}, \dots, \overline{p_m} \pm s_{p_m},) \quad (9)$$

Where:

$\overline{p_i}$ – average of the variable or parameter,

s_{p_i}- standard deviation of the variable or parameter.

Having this, the Two-point Technique does not require a large amount of calculations, but only 2^m simulations. The simulations are done based on known or assumed to be known average values and standard deviation values for each of the parameters encumbered with uncertainty. In the Two-point Technique standard deviation is a measure of the data uncertainty of the random variable. For all the points of a modelled area (catchment) as well as for the entire set time, the calculations must be done on the chosen variables of the dynamic system using a model and assuming that every data (p_i, $i = 1,\dots,N$) is encumbered with an error (uncertainty) assuming one of the two values: $\overline{p_i} + s_p$ or $\overline{p_i} - s_p$. It's worth noting that for a small number of parameters or variables encumbered with uncertainty leads to a small amount of calculations required. For example, if the analysed system has two parameters encumbered with uncertainty, the amount of simulations required is equal to four. However, if the analysed system has more parameters encumbered with uncertainty, the required number of simulations to run rapidly increases, for instance with ten parameters, one must conduct 2^{10} = 1024 simulations. Hence, the method is very efficient for models with a small number of variable parameters.

5.1. Analysis of the model sensitivity to data uncertainty

5.1.1. Methodology

Based on the case study presented in section 3. Watershed Bounded Network, the Two-point Technique for the model will be done based on the assumptions there are four main parameters encumbered with uncertainties, as presented in table 12. Overall, the total amount of simulations that must be done is equal to 16 runs.

Table 12: Two-point method - four parameters OBARCZONE with uncertainty.

Set	Number of simulations	Parameters ridden with uncertainty
1	2 (2^1)	imp%
2	4 (2^2)	imp%, IL_{perv}
3	8 (2^3)	imp%, IL_{perv}, IL_{imp}
4	16 (2^4)	imp%, IL_{perv}, Il_{imp}, CL_{perv}

Where:

imp% - percentage of impervious cover over the catchment,

IL_{perv} – initial losses generated over pervious surfaces,

IL_{imp} – initial losses generated over impervious surfaces,

CL_{perv} – continuous losses generated over pervious surfaces.

Table 13 presents the assumed values for the chosen parameters – their average and standard deviation.

Table 13: Two-point method parameter values (average assumed and standard deviation.)

Parameter	Value	Parameter	Value
imp%	20.00	$s_{imp\%}$	13.00
IL_{perv}	21.84	s_{ILperv}	18.88
Il_{imp}	1.00	s_{ILimp}	1.00
CL_{perv}	1.20	s_{CLperv}	1.20

Following the choice of parameters and their values, table 14 presents all the required combinations to run the simulations of the model in accordance with the Two-point Technique.

Table 14: Two-point method approach - four parameters encumbered with uncertainty.

Combination	Parameter			
	Imperviousness	Pervious losses	Impervious losses	Losses Rate
1	imp% + $s_{imp\%}$	IL_{perv} + s_{ILperv}	IL_{imp} + s_{ILimp}	CL_{perv} + s_{CLperv}
2	imp% + $s_{imp\%}$	IL_{perv} + s_{ILperv}	IL_{imp} + s_{ILimp}	CL_{perv} - s_{CLperv}
3	imp% + $s_{imp\%}$	IL_{perv} + s_{ILperv}	IL_{imp} - s_{ILimp}	CL_{perv} + s_{CLperv}
4	imp% + $s_{imp\%}$	IL_{perv} + s_{ILperv}	IL_{imp} - s_{ILimp}	CL_{perv} - s_{CLperv}
5	imp% + $s_{imp\%}$	IL_{perv} - s_{ILperv}	IL_{imp} + s_{ILimp}	CL_{perv} + s_{CLperv}
6	imp% + $s_{imp\%}$	IL_{perv} - s_{ILperv}	IL_{imp} + s_{ILimp}	CL_{perv} - s_{CLperv}
7	imp% + $s_{imp\%}$	IL_{perv} - s_{ILperv}	IL_{imp} - s_{ILimp}	CL_{perv} + s_{CLperv}
8	imp% + $s_{imp\%}$	IL_{perv} - s_{ILperv}	IL_{imp} - s_{ILimp}	CL_{perv} - s_{CLperv}
9	imp% - $s_{imp\%}$	IL_{perv} + s_{ILperv}	IL_{imp} + s_{ILimp}	CL_{perv} + s_{CLperv}
10	imp% - $s_{imp\%}$	IL_{perv} + s_{ILperv}	IL_{imp} + s_{ILimp}	CL_{perv} - s_{CLperv}
11	imp% - $s_{imp\%}$	IL_{perv} + s_{ILperv}	IL_{imp} - s_{ILimp}	CL_{perv} + s_{CLperv}
12	imp% - $s_{imp\%}$	IL_{perv} + s_{ILperv}	IL_{imp} - s_{ILimp}	CL_{perv} - s_{CLperv}
13	imp% - $s_{imp\%}$	IL_{perv} - s_{ILperv}	IL_{imp} + s_{ILimp}	CL_{perv} + s_{CLperv}
14	imp% - $s_{imp\%}$	IL_{perv} - s_{ILperv}	IL_{imp} + s_{ILimp}	CL_{perv} - s_{CLperv}
15	imp% - $s_{imp\%}$	IL_{perv} - s_{ILperv}	IL_{imp} - s_{ILimp}	CL_{perv} + s_{CLperv}
16	imp% - $s_{imp\%}$	IL_{perv} - s_{ILperv}	IL_{imp} - s_{ILimp}	CL_{perv} - s_{CLperv}

Having the prepared combinations, the base model must be ran using the appropriate values (table 17, 11. Annex). Then, the results generated for each of the sixteen simulations (peak flow values at the outlet of the culvert *Q*), must be averaged as presented in equation 8. To calculate variance, the following is applied:

$$V_Q = \sum(Q^2) - \left(\sum(Q)^2\right) \tag{10}$$

Where:

V_Q – discharge variance [m³/s]

Q – discharge] [m³/s]

Next, standard deviation is calculated as the square root of variance:

$$s_Q = \sqrt{V_Q} \tag{11}$$

Where:

s_Q – square root of discharge variance [m^3/s]

V_Q – discharge variance [m^3/s]

The result may also be presented as the variation coefficient expressed as a percentage:

$$CV = \frac{s_Q}{\bar{Q}} \cdot 100\% \tag{12}$$

Where:

CV – variation coefficient [-]

s_Q – square root of discharge variance [m^3/s]

$\bar{Q}$ – mean discharge [m^3/s]

Finally, the result may be presented in the form:

$$\boldsymbol{Q = \bar{Q} \pm s_Q}$$

5.1.2. Results

Table 15 below, presents the results for each of the 16 simulations ran for each of the observed annual recurrence intervals. Figure 19 presents the same data in the form of a graph, where the peak discharge values are presented for each ARI.

Table 15: Outflow peak flow values [m^3/s] generated by the model for the parameter combinations presented in table 14.

Combination	ARI						
	1	2	5	10	20	50	100
1	6.72	12.93	21.30	26.06	32.88	38.73	42.86
2	7.09	13.83	22.15	26.77	33.59	39.15	43.40
3	6.99	12.98	21.37	26.24	33.06	38.83	42.97
4	7.12	13.87	22.21	26.93	33.77	39.25	43.50
5	19.00	23.75	31.74	37.26	41.11	46.09	49.07
6	19.58	24.68	32.81	37.76	41.71	46.67	49.54
7	19.17	23.91	31.92	37.37	41.24	46.22	49.16
8	19.68	24.84	32.99	37.87	41.84	46.75	49.63
9	3.86	10.02	18.74	21.14	27.11	35.38	39.10
10	5.27	22.68	30.34	36.30	39.89	44.72	48.10
11	3.87	10.03	18.75	21.16	27.14	35.41	39.12
12	5.27	11.61	19.83	22.23	28.29	36.21	39.89
13	17.19	22.24	29.82	36.00	39.50	44.30	47.80
14	18.83	23.59	31.36	36.64	40.33	45.25	48.46
15	17.22	22.27	29.85	36.02	39.53	44.33	47.82
16	18.86	22.68	30.34	36.30	39.89	44.72	48.10

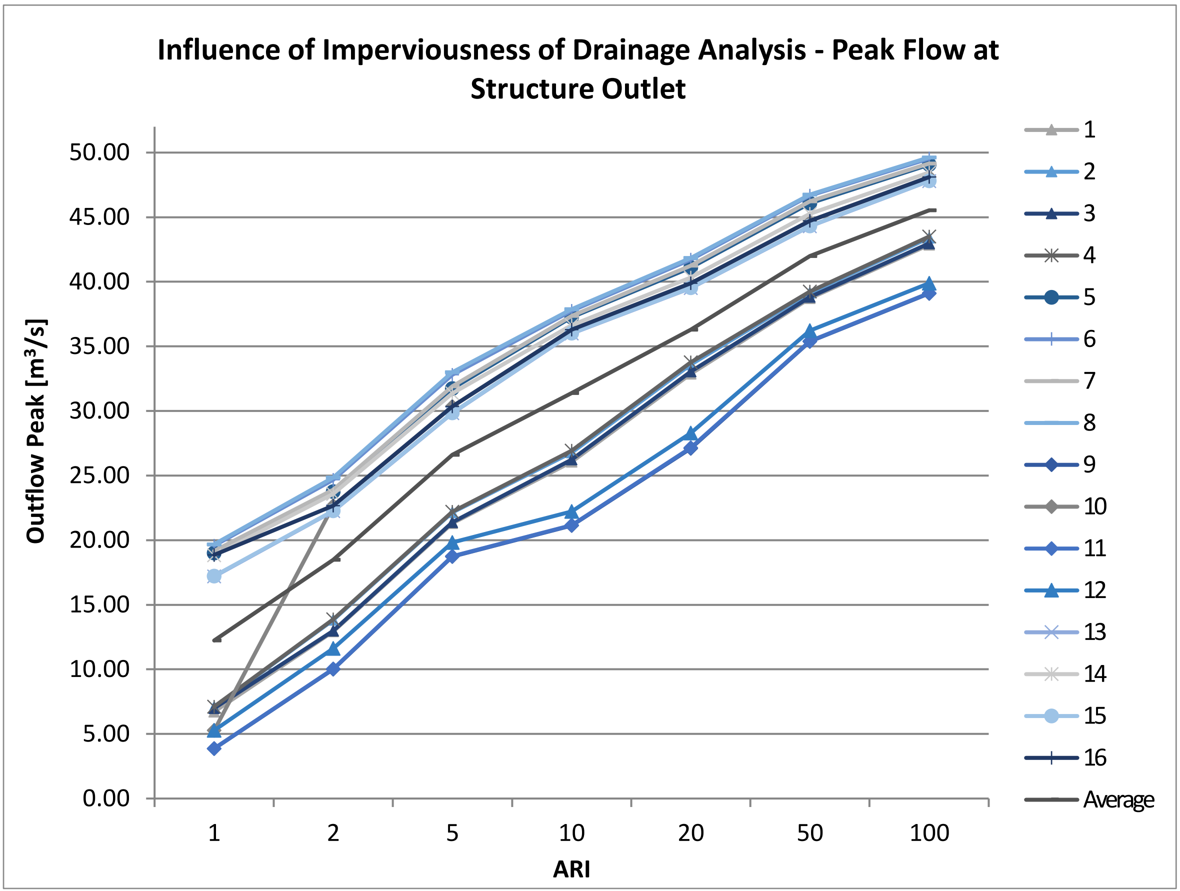


Figure 19: Peak flow values for different annual reoccurrence intervals for each of the model runs done according to the parameter combinations in table 14.

5.1.3. Discussion

The results generated in this part of the study cannot be compared to the base values, as the Two-point Technique allows to assess how prone the model is to the uncertainty of its parameters. This in turn may be related to how the model is prone to changes in imperviousness which is treated as one of the parameters. When looking at figure 19, the peak discharge values differ from each other significantly. It is important to remember that in each of the simulations, there are either no losses (pervious or impervious surface), no losses rate or the pervious areas are very small. For instance, as in case of trial 10, the peak values quickly escalate from 1 year ARI to 2 year ARI discharge. This may be caused by the combination of parameters, where the impervious area extent is small, initial losses over pervious surfaces are very high but there are no continuous losses.

In order to analyse the results obtained, the parameters of average, variance, standard deviation and the variation coefficient are calculated and presented in table 16 for each ARI. Based on the table, it is clear that the effect of uncertainty of the model decreases with increasing storm intensity – both standard deviation and the variance coefficient decrease significantly with increasing ARI. Overall the error encumbered on the peak flow values for different ARI's is as follows:

$$Q_{peak,\ 1-year\ ARI,} = 12.23 \pm 6.56\ \frac{m^3}{s}$$

$$Q_{peak,\ 2-year\ ARI,} = 18.49 \pm 5.70\ \frac{m^3}{s}$$

$$Q_{peak,\ 5-year\ ARI,} = 26.60 \pm 5.42\ \frac{m^3}{s}$$

$$Q_{peak,\ 10-year\ ARI,} = 31.38 \pm 6.43\ \frac{m^3}{s}$$

$$Q_{peak,\ 20-year\ ARI,} = 36.30 \pm 5.24\ \frac{m^3}{s}$$

$$Q_{peak,\ 50-year\ ARI,} = 42.00 \pm 4.12\ \frac{m^3}{s}$$

$$Q_{peak,\ 100-year\ ARI,} = 45.53 \pm 3.77\ \frac{m^3}{s}$$

Table 16: Two-point Technique parameter evaluation.

Combination	ARI						
	1	**2**	**5**	**10**	**20**	**50**	**100**
Average:	12.23	18.49	26.60	31.38	36.30	42.00	45.53
Variance:	42.97	32.49	29.36	41.32	27.42	17.01	14.21
SD:	6.56	5.70	5.42	6.43	5.24	4.12	3.77
CV [%]:	53.59	30.82	20.37	20.49	14.42	9.82	8.28

It's worth noting, that although the influence of uncertainty on different storm magnitude decreases, the peak discharge value rapidly increases. In case of the 2-year ARI, 5.70 m^3/s accounts for over 50% of the flow value, while for 20 year ARI, 5.24 m^3/s accounts for only 14.42% of the flow. Additionally, the uncertainty for a 10-year ARI suddenly increases, defying the overall trend of increasing storm intensity with decreasing dependence. In order to understand this behaviour, more studies on a bigger pool of data should be conducted.

6. Conclusions

Sensitivity analysis is a recommended step when studying a hydrologic model, as it gives insight into the degree to which the model outputs are influenced by uncertainty. Implementing uncertainty analysis is often impractical and time consuming, with a large amount of computations required. Point Estimate methods are therefore practical in general analysis, as they are not computation-heavy and are simple to analyse. This paper examined the influence general parameter uncertainty has on the model outputs as well as the influence of one parameter on the results generated by the model.

Overall, both studies have shown that higher intensity storms are influenced less by the parameters on the model outputs. In the parameter evaluation part of the study, it has been indicated that the model outputs are sensitive to the parameter of imperviousness, however, if the value of the parameter remains within realistic bounds for the given catchment, the change in the discharge is not that different from the base values, as presented in table 4. Increasing the storm intensity diminishes the effect imperviousness has over the catchment. Of course, the analysis conducted in this paper is still a very conservative and theoretical. The impervious cover is spread evenly throughout the entire catchment, which is unrealistic, but is one of the assumptions made in the study (also requested by the client in the initial analysis). The location and distribution of the urbanized, impervious areas may have significant influence on the flow throughout the catchment and could be evaluated in further studies.

In the Two-Point Technique analysis, once again the model outputs are less influenced by uncertainty in cases for larger storm intensities. However, in this case, it's the peak discharge value that rapidly increases while the uncertainty decreases slightly. This gives an impression that the variance coefficient decreases rapidly with increasing storm intensity, despite the uncertainty remaining very similar compared to quickly increasing discharge values – uncertainty decreases by only 2-3 m^3/s overall. Losses in the study are another crucial element, and figure 19 presents how the reacts to small changes in its settings. Removing any impervious cover over the catchment leads to the removal of any losses that could have occurred over those areas.

It is important to take into consideration, that the catchment investigated is located in an area with little seasonable variation, as presented in figure 6. However, studies have shown that precipitation intensity increases, but the events occur much less frequently, creating long dry periods in-between. This also could influence the model and should be taken into account. The model uses storm design data from 1987, while there is a newer version available according to the ARR and BoM, one which takes rainfall variation due to climate change into account.

For the future, several aspects of this study could be improved to provide a more accurate sensitivity analysis. Firstly, impervious cover should be assigned according to its realistic spread. Secondly, the model should be ran on the 2016 ARR storm design criteria, which include climate change variations within them. The results of the model should be compared to real life measurements, and not to a version of the model cleared as validated, if possible. Other loss models could be examined in WBNM and their influence on the model outputs could be studied.

Overall, based on this study it is possible to say that the initially provided conservative model is sufficient enough to generate results indifferently of the impervious cover extent in the investigated catchment. Combining the analysis conducted in this paper with a hydraulic model would provide the information on how much would the water level by the culvert change when taking into account the increase in peak discharge caused by increased imperviousness. Finally, based on the Two-Point Technique, it is visible that the model is prone to uncertainties related to its input parameters however small they may be.

7. Literature

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8. Table Index

9. Figure Index

10. Abbreviation Index

ARI – Annual Recurrence Interval

ARR – Australian Rainfall and Runoff

AWRA-L – Australian Water Resource Assessment

BoM – Bureau of Meteorology

CL – Continuous Losses

CL_{perv} – continuous losses generated over pervious surfaces.

CSIRO – Commonwealth Scientific and Industrial Research Organisation

DEM – digital elevation model

EIA – effective impervious area

EY – exceedances per year

FOSM – First-order second-moment

IFD – intensity frequency duration coefficients

IL – Initial Losses

IL_{imp} – Initial Losses from impervious surfaces

IL_{perv} – Initial Losses from pervious surfaces

ILSAX – ILLUDAS-SA with something extra (ILLUDAS-SA-eXtra)

Imp% - percentage of impervious cover over the catchment

LAI – Leaf Area Index

NSE – Nash-Sutcliffe efficiency

NSW – New South Wales State

PE – point estimate

PBIAS – Percent Bias

r – Pearson correlation coefficient

RAFTS – Runoff Analysis and Flow Training Simulation

RMSE – Root Mean Square Error

s_{CLperv} – standard deviation of percentage of continuous losses generated over pervious surfaces.

$s_{Imp\%}$ - standard deviation of percentage of impervious cover over the catchment

s_{ILimp} – standard deviation of percentage of initial Losses from impervious surfaces

s_{ILperv} – standard deviation of percentage of initial Losses from pervious surfaces

SWMM – Storm Water Management Model

ULX – underline crossing

WBNM – Watershed Bounded Network Model